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Journal of the prestressed concrete institute

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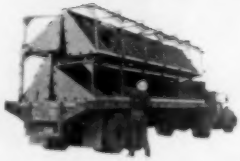

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Vol. 5, No. 4

JOURNAL

OF THE PRESTRESSED CONCRETE INSTITUTE

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Prestressed Concrete Trestle Saves Railroad Over $60,000

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Prestressed concrete piles and deck units were used in the construction of a new Atlantic Coast Line span in South Carolina. On this project, one of the first major prestressed concrete railroad trestles in the U. S., it was estimated that over $60,000 was saved by choosing the prestressed units over conventional concrete construction.

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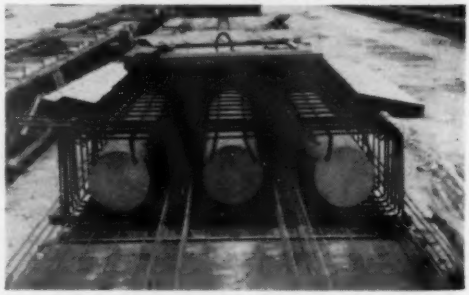

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This photograph shows the manufacture of I-beam bridge members for the Lomas Interchange of the Coronado Freeway, at Albuquerque, New Mexico. The manufacturer, American-Marietta Company, Concrete Products Division, used Bethlehem 7/16-in. Stress-Relieved Strand in making members from 61 ft to 75 ft long, to meet H-20-S16 loading.

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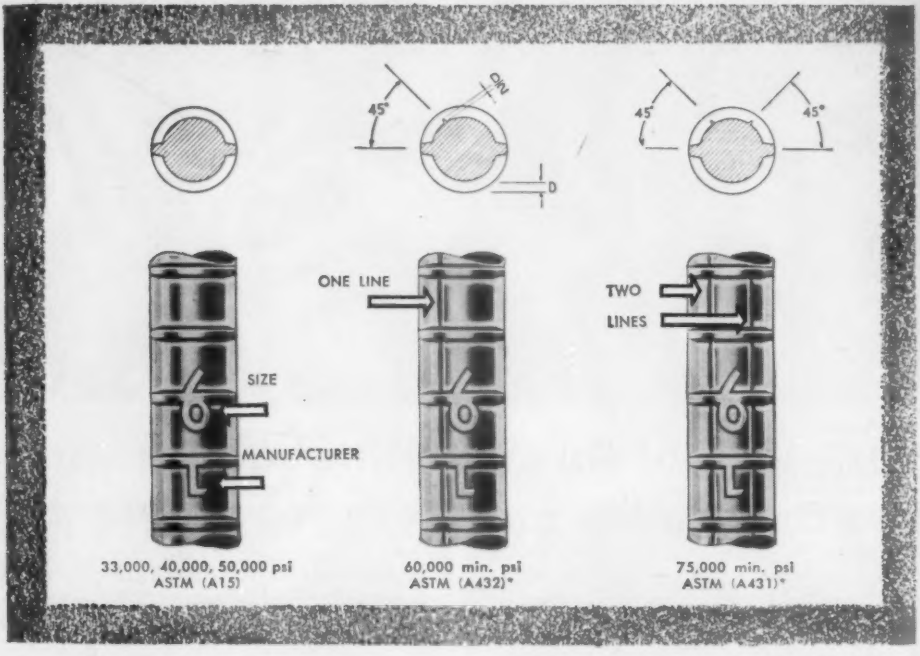

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Contractor: Farsura, both of Milan, Italy.

Flexible concrete plates drape-shaped to line canal bed

New technique uses prestressed POZZOLITH concrete

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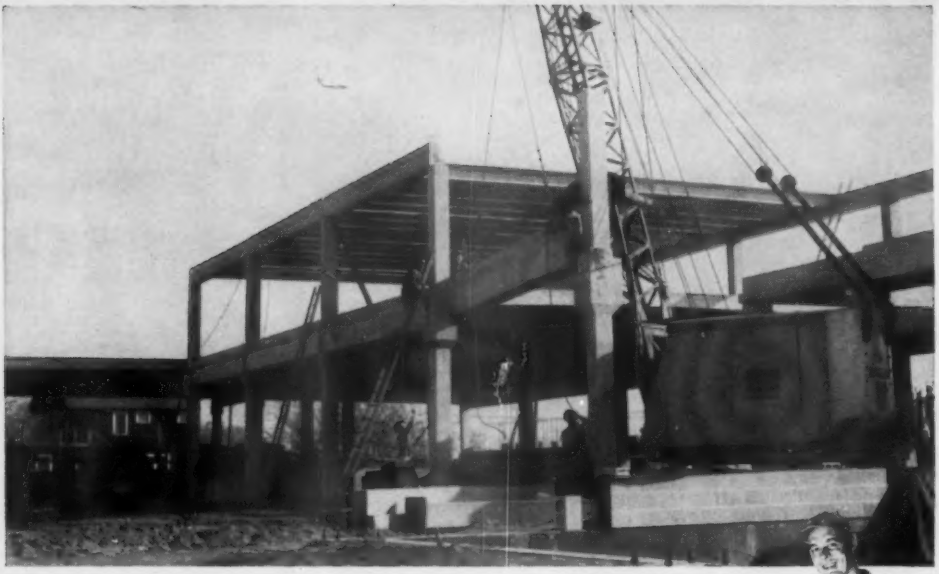

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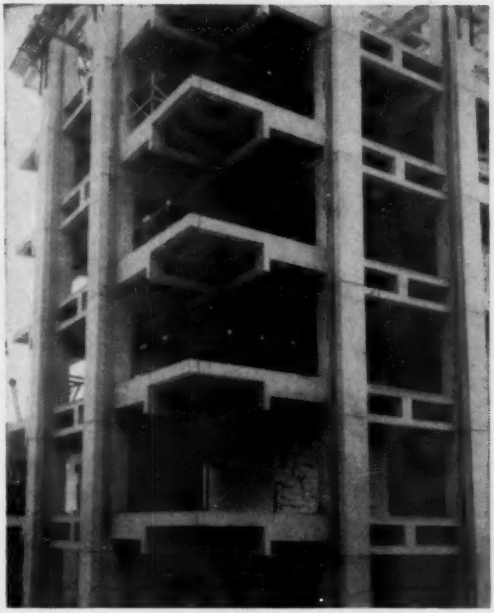

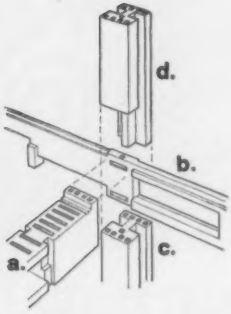

Example of the complex interlocking. Vierendeel truss (a) and spandrel (b) rest on inside flange of H-column segment (c), with notched end of next H-segment (d) completing the assembly.

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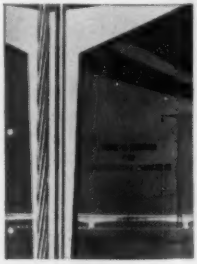

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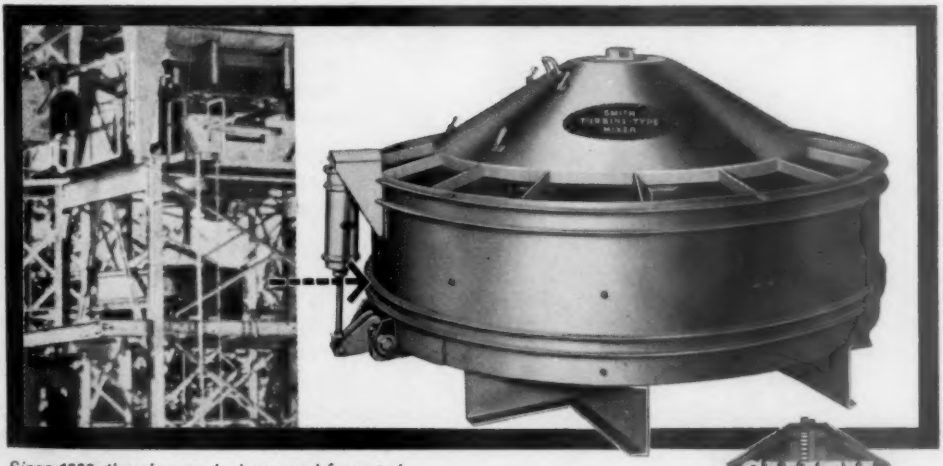

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GUEST EDITORIAL

The great development and expansion in the use of prestressed concrete in the United States in the past decade has indeed been sensational. It was slow in coming but when finally it came, it came like a flood from a bursting dam, sweeping all before it. Now it is established by its proven worth and accomplishments.

One of the changes that prestressed concrete has brought about is its own substitution for poured-in-place concrete. For many uses the precast, prestressed members are so much more economical than the in situ alternate—shored, formed, reinforced, concreted, finished, cured and finally stripped—that the latter lacks all excuse for being. Why laboriously and expensively do that which can be accomplished by another method with facility, dispatch and at a lesser cost?

There is a further advance which should be made, at least in the field of bridges. This is the substitution of precast slabs for poured-in-place roadway deck slabs. The handicap of weight from which all prestressed members suffer should be minimized by keeping beam sections to their irreducible least and precast slabs should span between them. Whether these slabs be merely precast or be precast and prestressed is to be determined by conditions prevailing. But there is no escaping the fact that field construction is inherently more costly than that of the products plant. Therefore to combine precast slabs with precast, prestressed beam stems and to reduce field construction to the assembling of precast units and to the pouring of connecting joints is to bring the making of bridges to its lowest and most economical terms.

JOURNAL in brief

Engineering Feats with Post-Tensioning 12
Jean Muller

A collection of European structures featuring unique applications of prestressing and ingenious methods of handling and erecting members.

Cylindrical Hollow Prestressed Concrete Piles 41
P. S. Heerema

The techniques used in fabrication and placing pretensioned piles up to 200 ft. long are discussed. The solution of problems concerning driving operations are also presented.

2 and 3 Wire Strand for Prestressed Concrete 48
R. O. Kasten

With increased automation and the requirements of smaller sections the author reports on the development of a 2 and 3 wire strand.

Neat Joints are Good Business 52
Laurence Cazaly

A truly practcial approach to that spot of concentrated trouble, the joint.

Prestressing and Its Function in Architecture 60
A. Kennerly

Included are this outstanding architect's views on the present uses of prestressed concrete. Also an example of what he refers to as prestressed concrete's expressionism. A series of questions are raised which represent the average architect's approach to prestressed concrete.

Behavior of prestressed Concrete Under Dynamic Loading 74
P. W. Marshall and A. M. Ozell

This investigation covers a subject on which little has been reported. It was carried on with a rather unusual prestressed concrete member—a diving board.

Europe's Contribution to Imaginative Engineering 85
Daniel C. Vandepitte

The techniques and advantages of employing exterior post-tensioned cables are discussed. Various statically indeterminate structures are also described with accompanying detailed drawings and photographs.

Engineering Feats With Post Tensioning

by Jean MULLER*

Paris, France

INTRODUCTION:

The purpose of this paper is to review some notable long span structures recently completed in Europe and the French Commonwealth. Particular emphasis is placed on certain features which might be of special interest to the American Engineer and Contractor.

CHAPTER I

Tancarville Bridge

Europe's longest suspension bridge was opened to traffic July 1959 at Tancarville, near Le Havre, over the Seine River. Although the main span of 3150 ft. (clear center span of 2000 ft.) is of steel construction, prestressed concrete was used extensively on the overall structure (see fig. 1).

Because of the layout of the river banks in the vicinity of the bridge, the suspension span could rest directly on the right bank cliff, in

*Assistant Chief Engineer
Entreprises CAMPENON BERNARD
Special Consultant to the Freyssinet Company Inc., New York

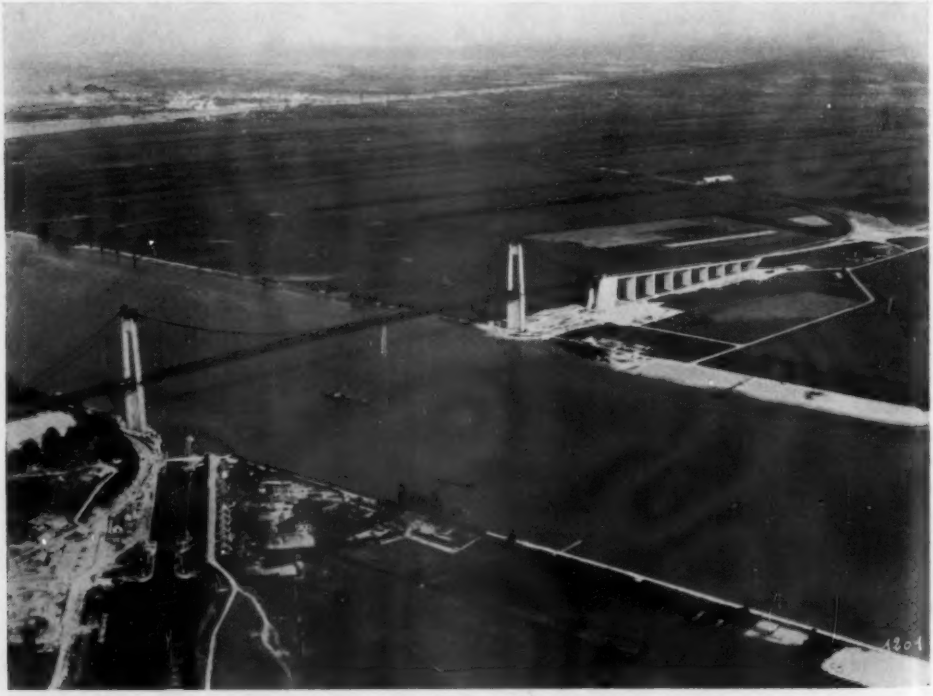


Fig. 1

spite of the 160 ft. clearance maintained above the river. On the left bank, however, natural ground was only a few feet above the river level and a long approach viaduct was required. Prestressing was used in the following parts of this bridge:

—Anchorage of the suspension cables in right bank,

—Counterweight of the suspension cables at left bank,

—Approach viaduct

1)—Right bank anchorage:

As shown in fig. 2, this structure consists of:

a)—Two concrete abutments providing a support to the side span of the main bridge together with carrying saddles for the directional change of the suspension cables.

b)—Two reinforced concrete chambers for spreading of the 60 individual strands (each made of 109 x .185″ dia. wires) of the two main cables. Total load of these is as follows: 17,600 short tons for loaded bridge with maximum temperature change and 12,500 tons at time of adjustment.

c)—Two huge prestressed concrete ties each 15 ft. high and 12 ft. wide with an access gallery running most of the length. The prestressing force of 10,600 tons in each tie is provided by 120 cables of 24 x 0.276 in. dia. wires.

d)—A cross beam connects the bottom ends of the ties and provides an area such as to limit the bearing pressure on the soil to a maximum of 6 tons per s.f.

2)—Left bank box anchorage:

This structure provides an anchorage for the suspension cables of the main span, carries the roadway and receives the first span of the approach viaduct (see fig. 3). It consists mainly of two vertical walls about 150 ft. high connected by the roadway slab and several transverse diaphragms. Each wall rests on a front concrete Freyssinet-type hinge and on a rear special bearing made of a shallow concrete rocker and two neoprene pads. The reaction of the suspension cables resolves with the dead weight of the structure and a concrete counterweight located at the rear into an inclined reaction transmitted to the foundation caissons by the front hinge. All members are post-tensioned. Analysis of the stresses in the main walls was made on a plastic scale model. The typical layout of the prestressing units in the wall is given in Fig. 4.

3)—**Approach viaduct** (see fig. 5 and 6).

Eight 164 ft. spans carry the roadway over the left bank to the main span. There are five 140 ton girders, 10 ft. deep in each span. All spans are simply supported and rest on hollow reinforced concrete cast in place piers. All girders were prefabricated in a yard located at the original ground level. Moving and lifting operations for one girder included (see Fig. 7 and 8):

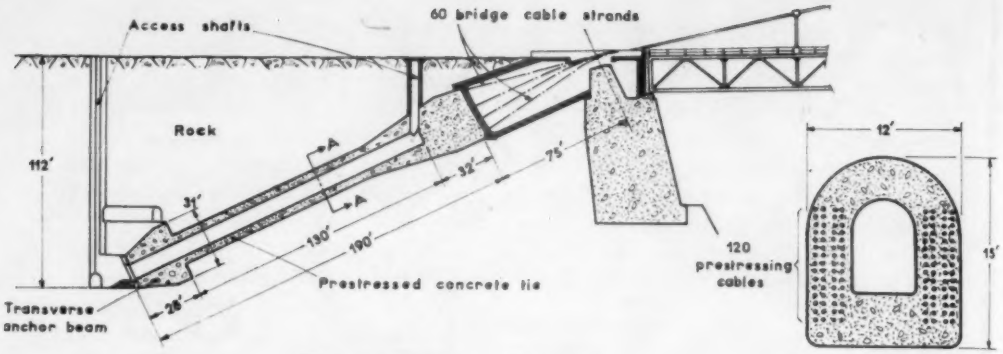
—The placing of girders on dollies, and their moving in two perpendicular directions to bring them to the foot of the supporting piers.

—Hoisting the girder along the piers with special steel rigs as shown in (Fig. 9).

—Placing the girder on top of the pier with rotating arm of the special rig (see Fig. 10).

—Transverse displacement of the girder to its final position.

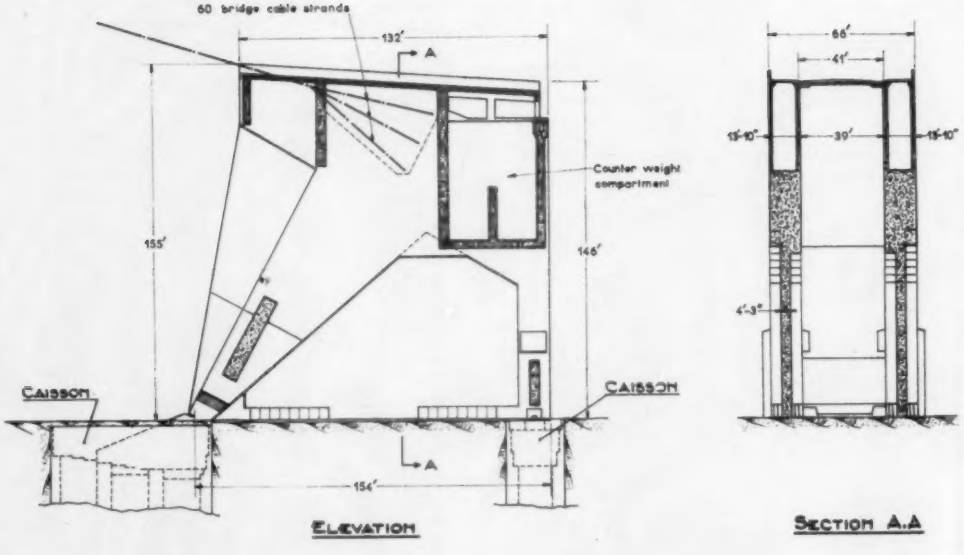
Suspension of the girder at both ends was achieved by the means of special cantilevers so as to provide for the highest factor of safety against lateral buckling during the lifting operations of such slender girders.



TANCARVILLE BRIDGE
RIGHT BANK TUNNEL ANCHORAGE

SECTION: A.A

Fig. 2

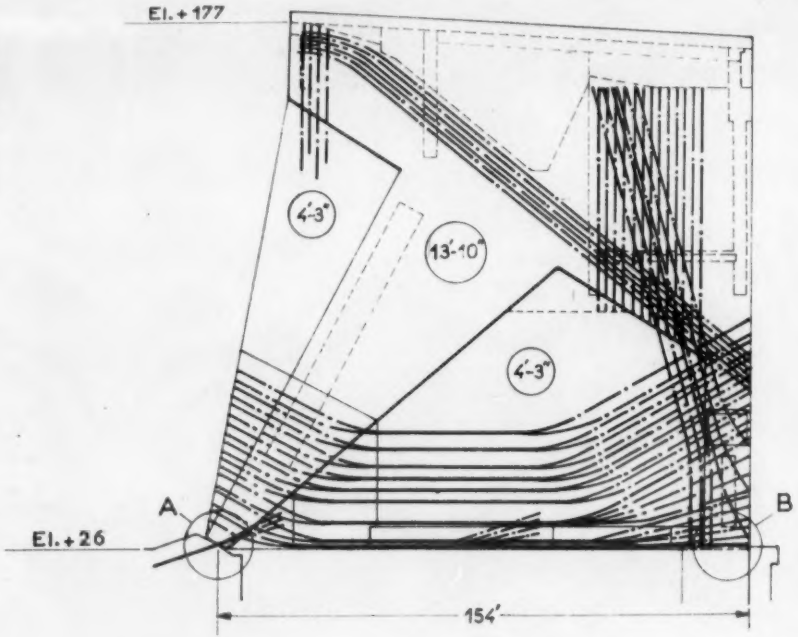


ELEVATION

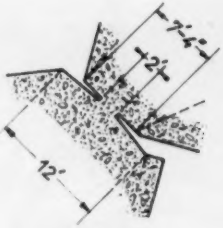
SECTION A.A

TANCARVILLE BRIDGE
LEFT BANK ANCHORAGE

Fig. 3



ELEVATION - PRESTRESSING LAYOUT



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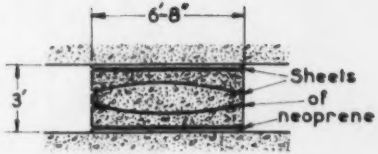


DETAIL "B"

TANCARVILLE BRIDGE
LEFT BANK ANCHORAGE

Fig. 4

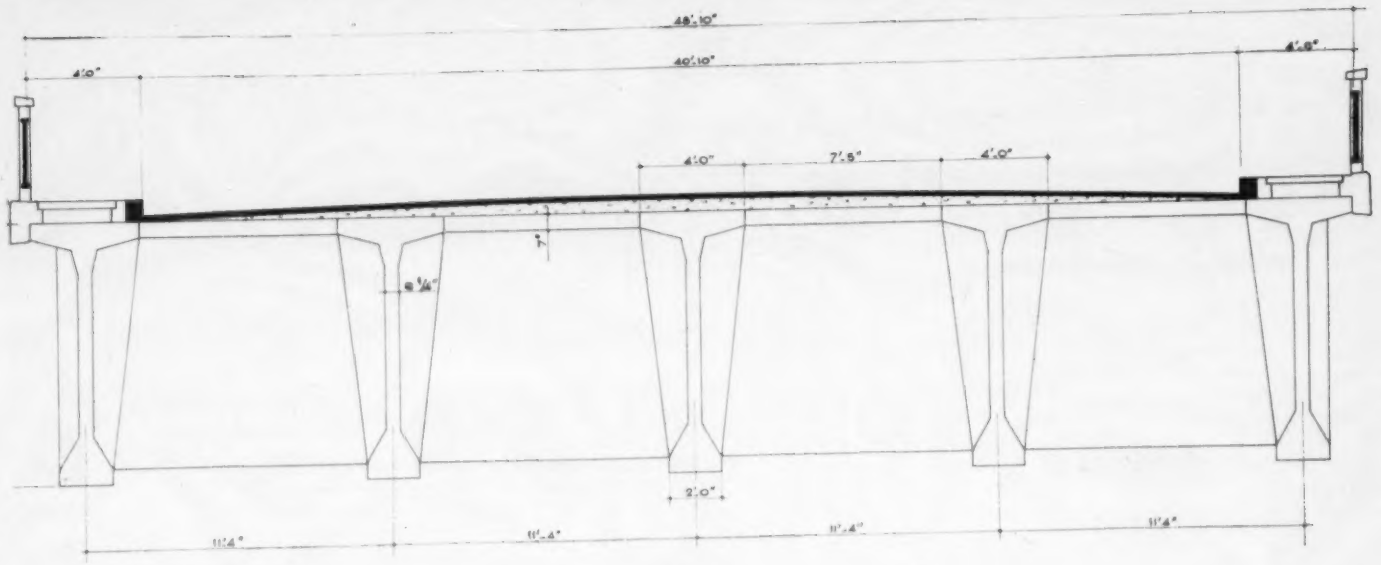

48'-10"
40'-10"
4'-0"
4'-0"
4'-0"
7'-9"
4'-0"
9 1/4"
2'-0"
11'-4"
11'-4"
11'-4"
11'-4"

Fig. 5

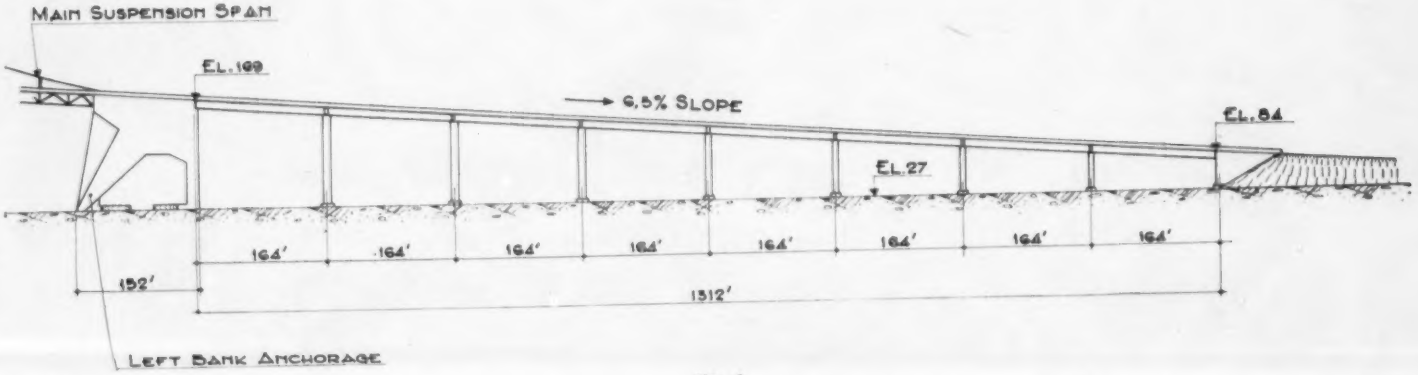

MAIN SUSPENSION SPAN
EL. 109
6.5% SLOPE
EL. 84
EL. 27
164'
164'
164'
164'
164'
164'
164'
164'
152'
1312'
LEFT BANK ANCHORAGE

Fig. 6

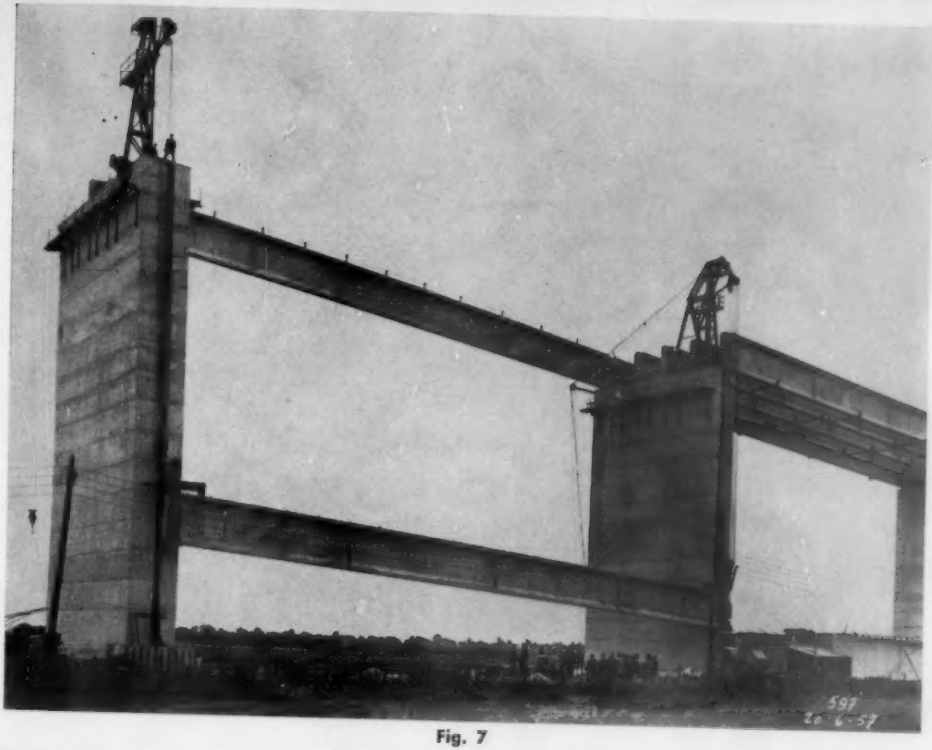


Fig. 7

Fig. 8

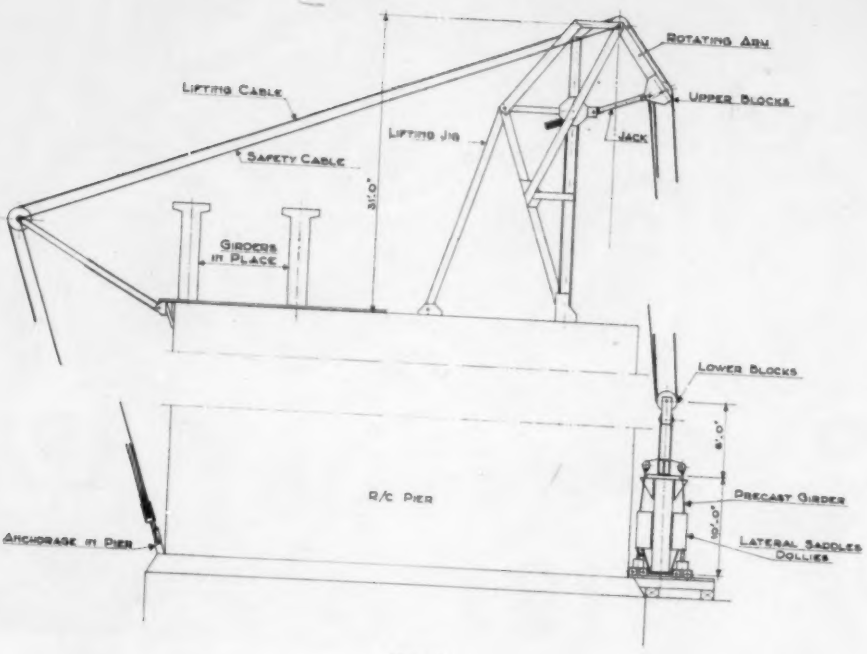
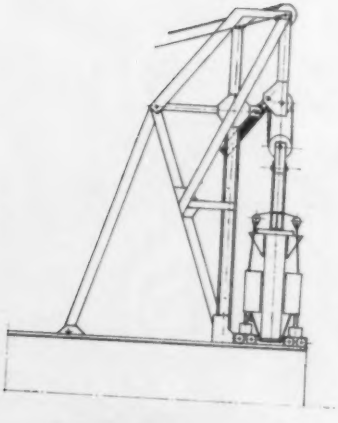
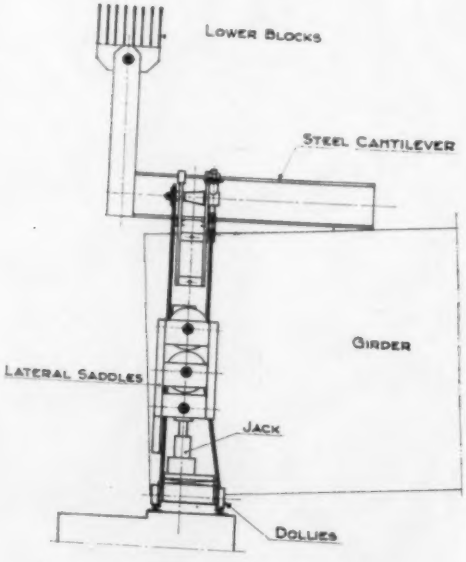


Fig. 9



PLACING GIRDER ON PIER
TANCARVILLE



DETAIL OF GIRDER SUSPENSION
TANCARVILLE

Fig. 10

During construction, a series of tests were carried out to determine the amount of friction of the prestressing units due to curvature and wobble effect. The following results were consistently obtained:

	Non Greased Cables	Greased with Soluble Oil
Friction Coefficient in Curve	f=0.22	f=0.18
Wobble Coefficient	K=0.001 per ft.	K=0.0003 per ft.

As a result, theoretical elongations were computed using the following values: f = .20 and K = .0003.

CHAPTER II

Handling And Placing Long Span Precast Bridge Members

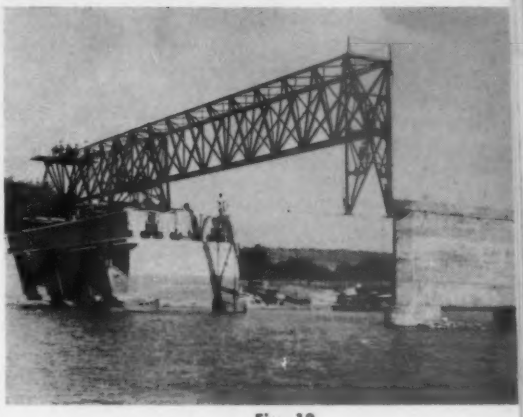
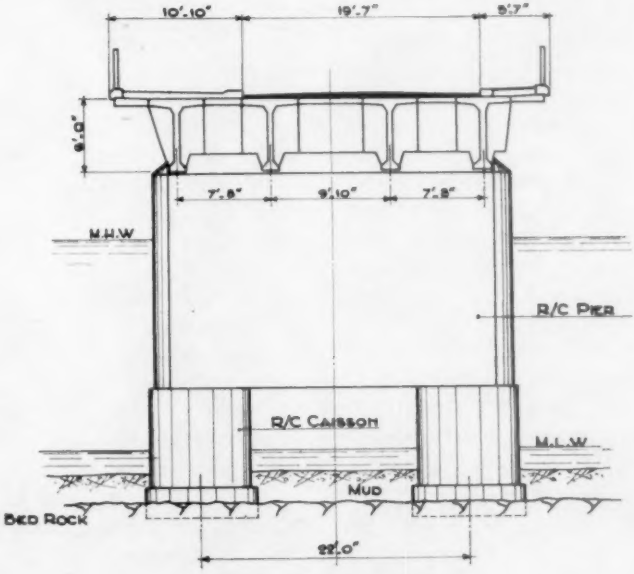


Fig. 12

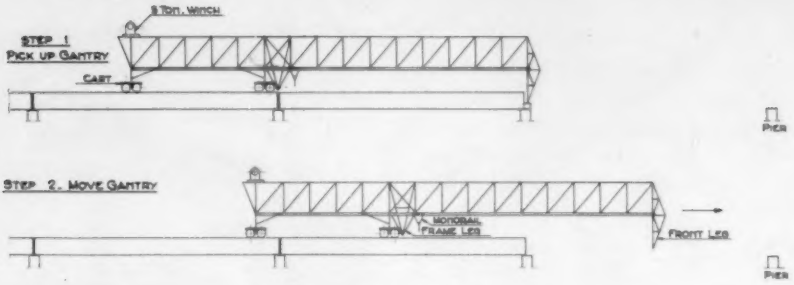
1)—**General:**

The approach spans of Tancarville Bridge are one of the many examples where precasting offers a decided advantage over cast-in place construction. However each large project warrants the use of special methods for handling precast members. Important factors in selecting the proper precasting



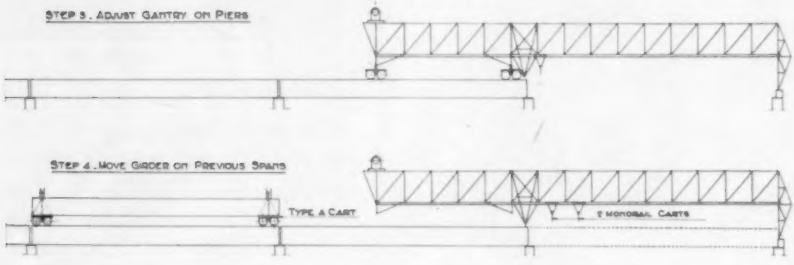
BAMAKO BRIDGE
TYPICAL SECTION

Fig. 11



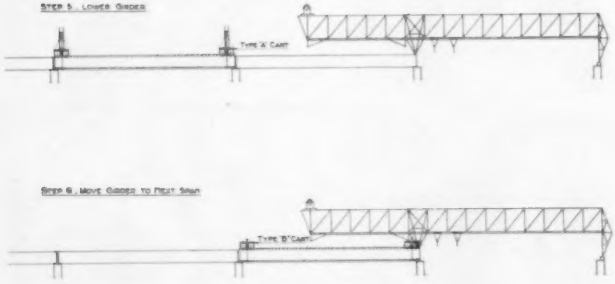
BAMAKO BRIDGE
GIRDER PLACING SCHEME (1) & (2)

Fig. 13



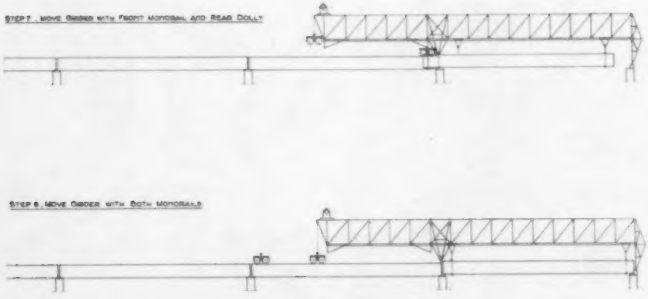
BAMAKO BRIDGE
GIRDER PLACING SCHEME (3) & (4)

Fig. 14



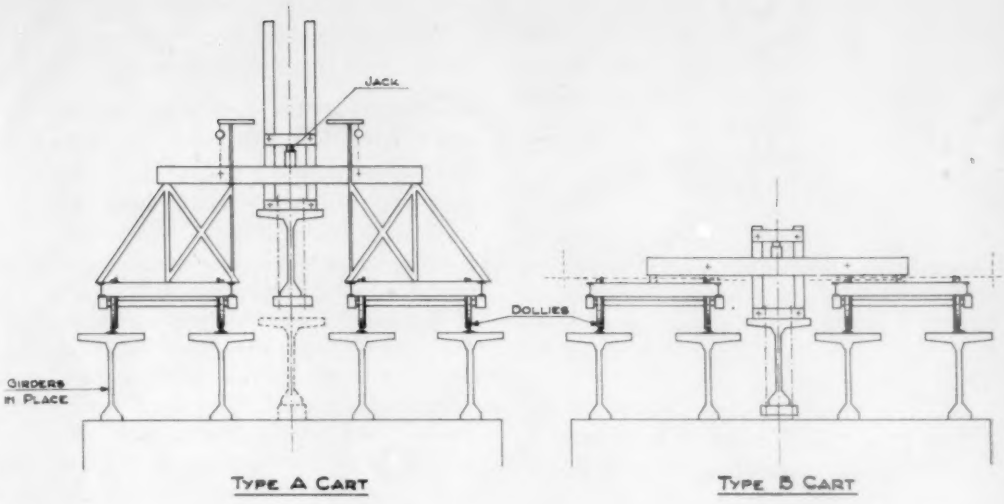
BAMAKO BRIDGE
GIRDER PLACING SCHEME (5) & (6)

Fig. 15



BAMAKO BRIDGE
GIRDER PLACING SCHEME (7) & (8)

Fig. 16



BAMAKO BRIDGE
GIRDER PLACING DETAILS (I)

Fig. 17

scheme include the following:

a)—Weight and length of precast units.

b)—Number of units to be handled.

c)—Special site conditions (such as bridge located over a navigable channel or over dry ground).

d)—Height of bridge above water or natural ground.

e)—Possible location of precasting yard.

Wherever practicable, the use of cranes or floating equipment provides usually the best answer. This particular subject will not be covered herein. In fact, this type of equipment as available in the U. S. includes probably higher capacity units than in other countries. Tampa Bay bridges and Lake Pontchartrain bridge may be mentioned as outstanding examples of the use of floating equipment. However, there are many instances where other methods must be used, because:

a)—Rental fees or write-off costs of operating one or two cranes are too high with regard to the advantages of precasting.

b)—Ground or floating cranes cannot be used on account of excessive weight and/or failure of access.

Two typical examples where special methods for handling and placing precast units were used are given hereafter.

2)—Use of a steel gantry at BAMAKO Bridge:

This bridge was constructed over the Niger River in former French Equatorial Africa, comprising thirty spans 95 ft. long each consisting of four girders 6′-0 deep, weighing 55 tons (see fig. 11). The river is barely navigable and subjected to important and sudden floods. Precast girders were placed with a steel gantry shown in (fig. 12). This gantry is moved along the center line of

the structure from span to span on carts travelling on rails. Concrete girders are placed according to the following step by step procedure (see fig. 13 through 16).

a)—Roll girder from precasting yard onto finished section of bridge.

b)—Pick-up girder with carts at each end and lower girder at finished level between girders of the previously placed span. Type A Cart (see fig. 17) is used for this purpose.

c)—Move girder longitudinally with type B carts at both ends to bring it under the monorail of the steel gantry.

d)—Switch front pick up point of girder from type B cart to front monorail dolly and continue longitudinal movement.

e)—Switch rear pick up of girder from type B cart to monorail movement.

f)—Skid girder transversely to final position.

Interesting features of this method include the following:

a)—The steel gantry is of triangular shape and the bottom chord provides a monorail track for the lifting devices.

b)—Type A cart may be turned into a type B cart easily and quickly. Furthermore, the same type of cart may be used for movements of the steel gantry.

c)—All carts distribute the loads of the steel gantry and/or concrete girders to the four girders of the previous spans. Relative girder spacing and width of top chord make it possible to lower moving girder between girders of the previous spans. (See fig. 18)

d)—Transverse sliding of girder is a very simple, quick and inexpensive operation when using a wooden skid as shown in (fig. 19).

The steel gantry is approximately 160 ft. long and weighs 22 tons.

3)—Placing by the cantilever method at OUED DJER bridges.

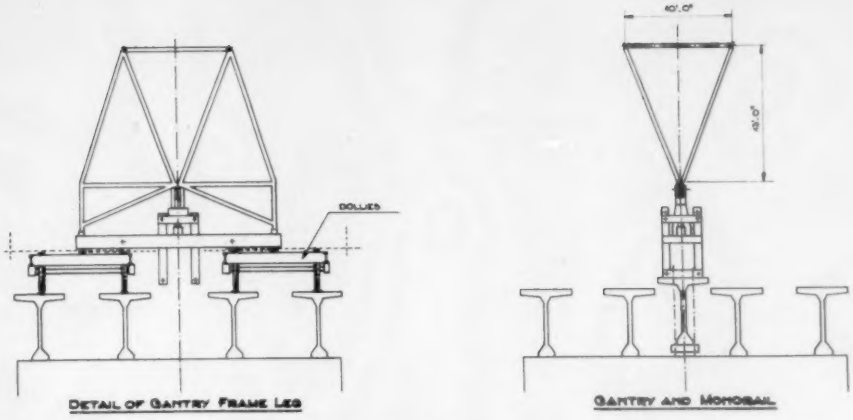
This project included the construction of 6 viaducts between Algiers and Oran, Algeria, each consisting of a variable number of spans ranging between 100 and 123 ft. The same cross section was used for all 140 girders (see fig. 20). These concrete girders with a maximum weight of 72 tons were cantilevered out from a steel tower. The tendency to overturning is resisted by another girder placed behind the tower and acting as a counterweight. The assembly comprising both girders and the steel tower is rolled onto the finished section of the bridge until the front end of the girder reaches the next pier (see fig. 21).

The girder is then picked up by two dollies in order to move it transversely to its final location. It is finally lowered into place by jacks and blocks attached to the dollies. The above method is used only for the first two girders in each span; the remaining girders are merely rolled over the previously placed girders and moved transversely by the same dollies. Placing the first two girders in one span required the following operations:

a)—Move girders from precasting yard on dollies over the preceding span.

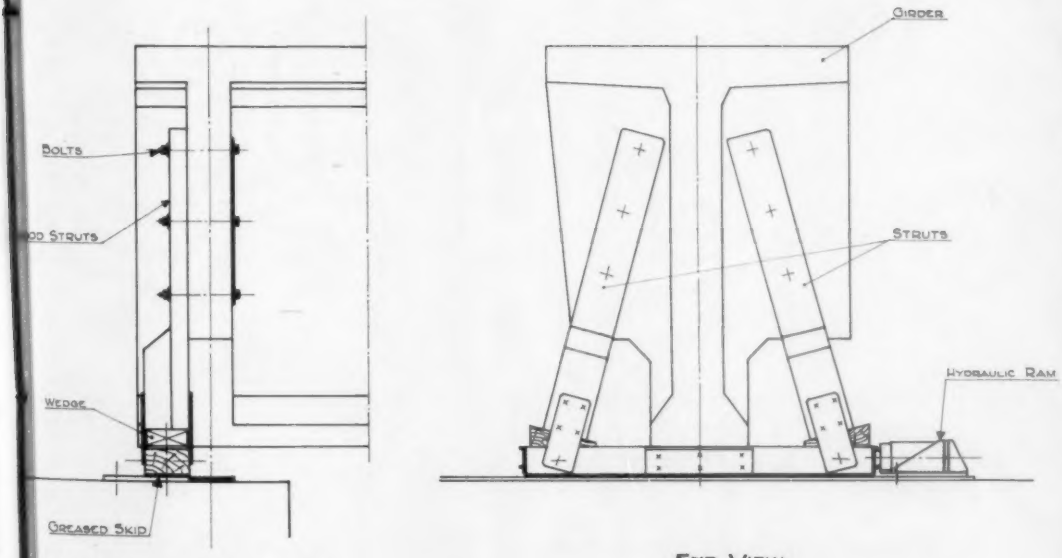
b)—Fasten girder to be placed to the tunnel dolly at rear end (so called because the girder passes through it), and to the front dolly at far end (a steel cantilever is provided for that purpose—see details in fig. 22).

c)—Place steel tower (disconnected from tunnel dolly) and counter-



BAMAKO BRIDGE
GIRDER PLACING DETAILS (II)

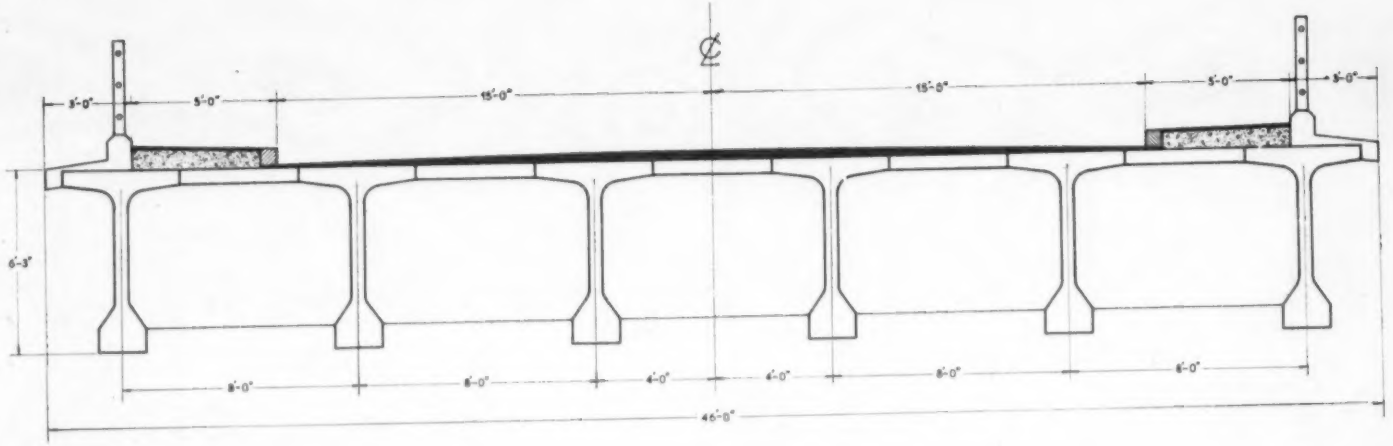
Fig. 18



BAMAKO BRIDGE

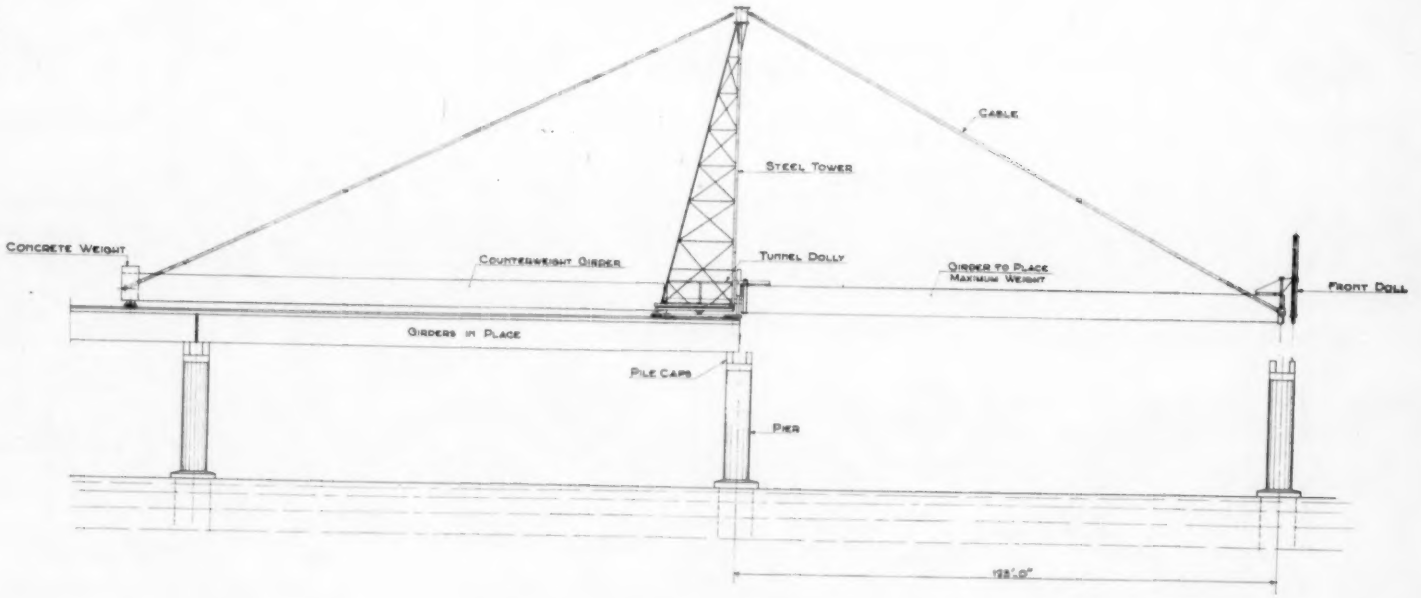
APPARATUS FOR TRANSVERSE SLIDING OF GIRDER

Fig. 19



OUED. DJER BRIDGES
TYPICAL SECTION OF 100 FT SPANS

Fig. 20



OUED DJER BRIDGES
GIRDER PLACING SCHEME

Fig. 21

weight girder with strut between both girders. Place tie cables at both ends of girders and over steel tower.

d)—Launching of girder. Both girders, the steel tower and its related equipment are moved longitudinally as one piece until the nose of the front girder reaches the center of the next pier. (fig. 23 and 24)

e)—Transverse displacement of girder;

The front dolly is connected to a lateral track which is lowered onto the cap. The counterweight girder and the steel tower are backed up so as to clear the way of the tunnel dolly.

Both dollies and the concrete girder are then moved transversely above its final position.

f)—Placing girder.

Both dollies are equipped with jacks permitting the girder to be lowered on the caps at the exact location.

g)—Launching second girder.

The procedure repeats itself exactly at the following exception. The front dolly is supported by the cap of the outward pier. The second girder is therefore launched without a front dolly. The latter is only used for transverse displacement.

h)—Placing other girders.

The first two girders are in place at the center of the span and are used as a bridge to carry the subsequent ones. Transverse movement and lowering into place make use of both tunnel dolly and front dolly. For a view of the completed structure (see fig. 25).

4)—Survey of various problems connected to the handling and placing of precast girders.

TANCARVILLE, BAMAKO and OUED DJER bridges exemplify various methods successfully used for placing long span prestressed concrete girders where prevailing conditions at the site did not permit the use of ground or floating cranes. Constant improvements are being brought about as more experience is gained in this field. In this respect, items of interest include:

a)—The recent use of carts on rubber tires in lieu of dollies on tracks for carrying precast girders.

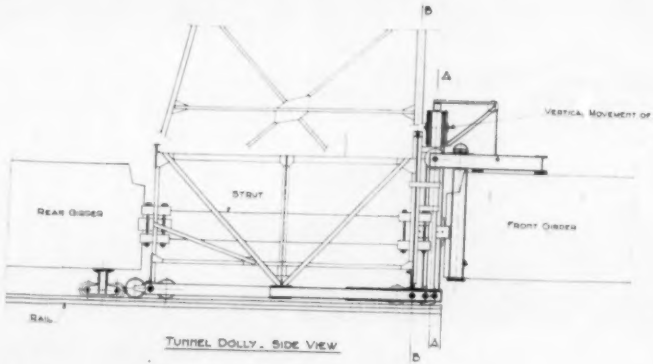
b)—The substitution of hydraulic rams to winches in the monorail carts to lower girders into position.

c)—The combination of a tower and tie cables with a monorail beam for very long spans instead of the more conventional steel gantry.

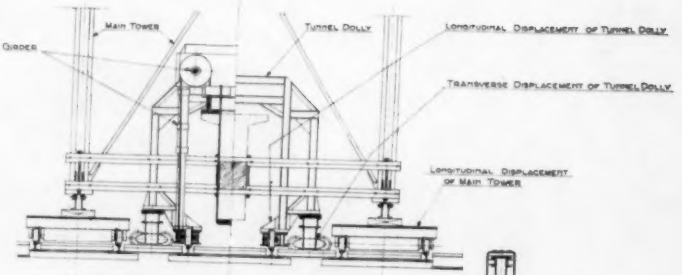
Regardless of the methods used for handling precast elements, particular care must be given to the elastic stability of members throughout the operations. Design must cover the following points:

a)—Choice of cross-section: A hollow section (possibly made up of two separate girders connected together for handling) provides a very stable member against lateral buckling. However such a section can seldom be used on account of its excessive weight, and the unsymmetrical "I" beam is almost exclusively selected. Lateral stability depends principally upon the width of the top chord.

b)—Means used for lifting girder. Devices used must be such as to avoid rotation of end sections; otherwise lateral buckling will occur much earlier. Whenever possible, the distance between pick up points should be chosen substantially smaller than the span length. Particular care must be given to the stress conditions

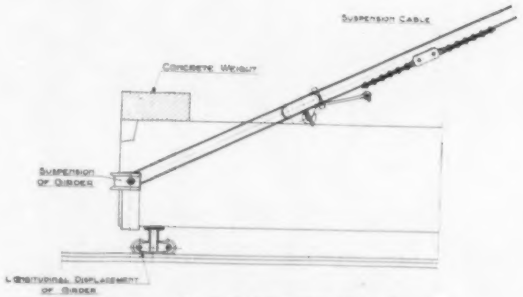


Tunnel Dolly - Side View



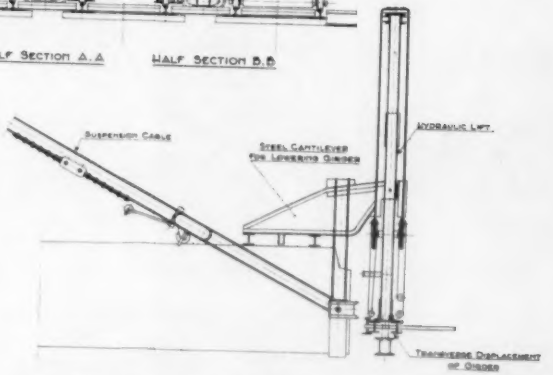
Half Section A.A

Half Section B.B



Rear End of Counter Weight Girder

Oued Djer Bridges
Girder Placing Details



Front End of Girder Moved to Place

Fig. 22

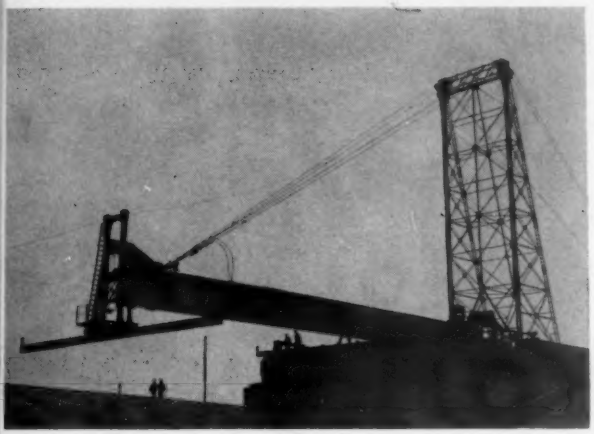


Fig. 23

at mid span and at the sections of the pick up points.

c)—Temporary bracings at supports and connections between girders should be provided as soon as girders are placed over the supports.

d)—Influence of wind loads and increase of lateral bending due to buckling tendency. The resistance of a beam to horizontal loads is greatly reduced for slender members on account of the interaction between lateral bending and lateral buckling.

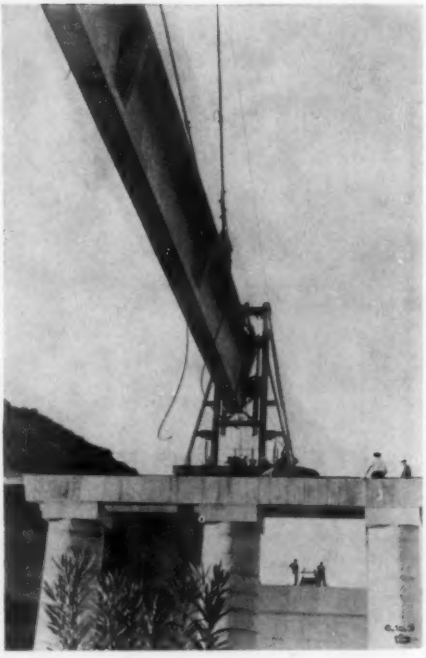


Fig. 24

Fig. 25

CHAPTER III

Some Other Notable Recent Bridge Structures

1)—Bridges at ORLY Airports

These bridges carry one aircraft runway, two taxiways and a parking area over the Paris-Riviera main highway which is located throughout the airport in a 25 ft. deep trench. Widths of these bridges along the highway are 1000 ft, 540 ft and 140 ft. Crosswise all bridges are of two unequal spans of 80 ft and 55 ft respectively (see fig. 26 and 27).

Unusual or interesting problems connected with this project include:

a)—Magnitude of live load:

In addition to standard highway loading, superstructure was to be designed for aircraft loading including a 240 ton plane with four - 27 ton closely spaced wheel loads and a 110 ton plane with 50 ton wheel loads all with a 300 psi tire pressure.

b)—Design of deck:

Because of the very heavy concentrated loads, girders were connected with stiff diaphragms (two in the longer and one in the shorter span), on order to distribute the loads to several girders.

Diaphragms were considered for design as beams on elastic foundation, the deflections of which are interrelated. For any loading arrangement these deflections are the solution of a differential equation of the twelfth order, which may be resolved into three separate equations of the fourth order. For a concentrated load applied over a given girder, the effect of the diaphragms is to distribute the loads to the three adjacent girders on each side. The maximum load supported by the center girder is only 30% of the applied load.

c)—Combined use of prestressing tendons and untensioned hard grade deformed bars in both girders and deck slab.

d)—Extensive use of neoprene bearing pads under center columns and over abutments. Consequently all expansion joints could be avoided even for the 1000 ft. wide bridge.

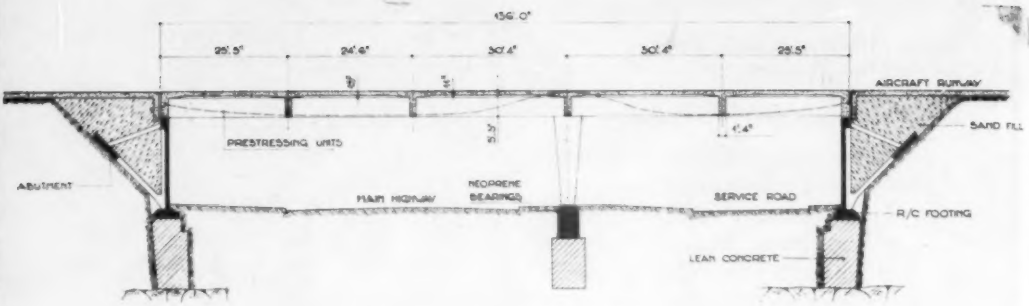
e)—Longitudinal prestressing units in deck slab were threaded through preformed holes in lengths up to 530 ft. and no difficulties were encountered in this operation. Figures 28 and 29 give two aspects of the completed bridges.

2)—Saint-Michael Bridge at TOULOUSE: (Figures 30, 31 and 32).

This bridge consists of five spans of 216 ft. each built over the Garonne River. Total width of the structure is 88 ft. with a 60 ft. roadway. Each span is made of 10 prestressed concrete double-hinged frames spaced 8′-3″ on centers (see figures 33 and 34).

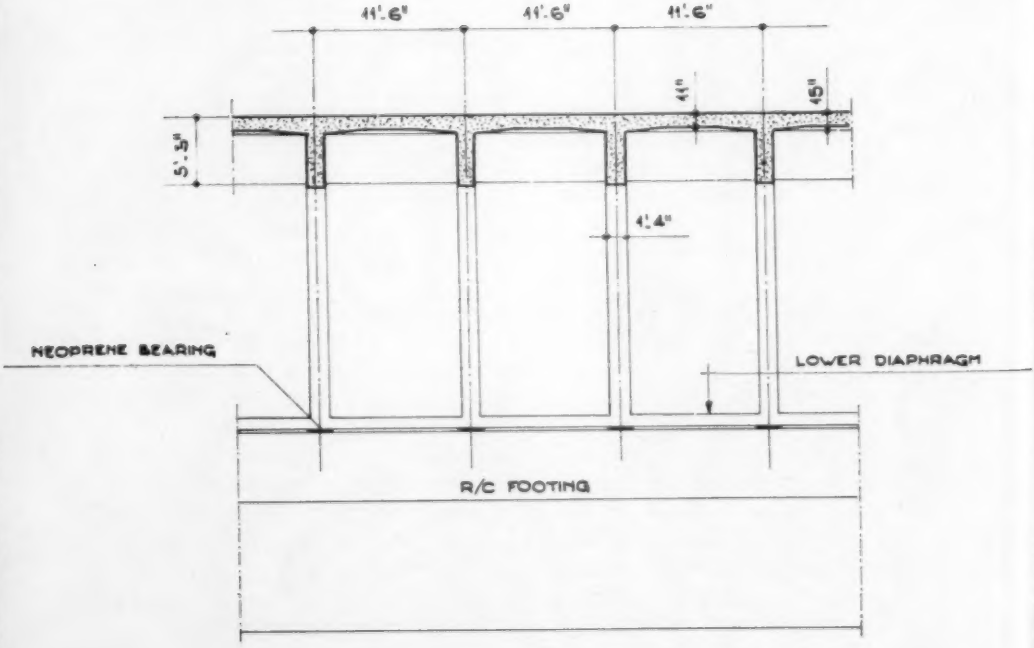
The old steel structure was used as a scaffolding for the new cast in place structure. The portal legs over each pier are cantilevered out on suspended forms tied to the existing masonry piers which were properly strengthened to resist the additional reactions. The corresponding construction procedure is outlined in fig. 35.

In order to insure the continuity of the deck over the piers, all girders are extended over the inclined portal legs with cantilever sections joined together at the center line of each pier. To reduce bending moments and consequent deflections in these rather slender cantilevers, a special connection between two abutting members has been provid-



BRIDGES AT PARIS-ORLY AIRPORT
TYPICAL CROSS SECTION

Fig. 26



BRIDGES AT PARIS-ORLY AIRPORT
PARTIAL LONGITUDINAL SECTION

Fig. 27

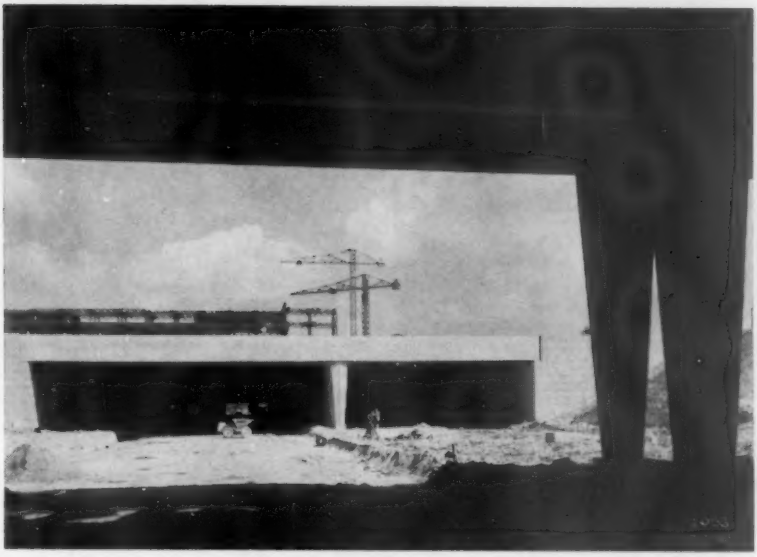


Fig. 28

ed. Such a connection using a neoprene bearing makes it possible to achieve continuity of the roadway without impairing thermal expansion.

3)—**Bridge at ABIDJAN, Ivory Coast:**

The main structure is a double decked, combined highway and rail-

Fig. 29

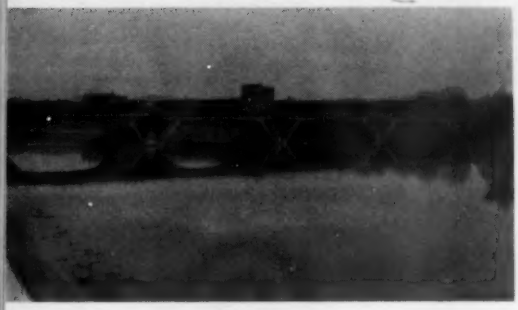


Fig. 30

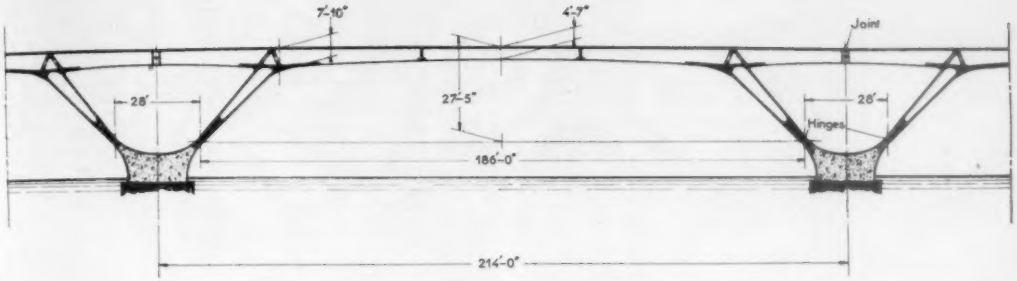
way bridge 1220 ft. long. Conventional type structures are provided at both ends for access and include clover leaf grade separations. The piers for the main structure are placed 52 ft. on centers. Superstructure for each of the eight spans consists of two tubular precast prestressed concrete beams weighing 880 tons each (see fig. 36 and 37).

The foundations are of unusual interest (see fig. 38). Average water depth is 33 ft. covering 50 to 65 ft. of soft mud, 70 ft. of hard mud underlaid by sand containing thin beds of hard clay. Therefore over the center section of the main bridge, the load bearing strata are located at depths averaging 170 ft. Foundations consist of prestressed concrete caissons floated into place resting on circular 54 in dia. piles ranging in length between 110 and 210 ft. with a maximum bearing capacity of 650 tons. Each of these shafts is made up of circular concrete pipe elements with a 4 in wall assembled together by post tensioning. Boring of the holes housing these shafts was made according to the Benito process. These shafts penetrate about 50 ft. into the sand stratum and additional lateral support is provided on two-20 ft. sections by inflation with cement grout of an expanding sleeve against the surrounding soil. This procedure exemplifies another aspect of prestressing. The inside space was then filled with concrete so as to provide a solid section.

Fig. 31

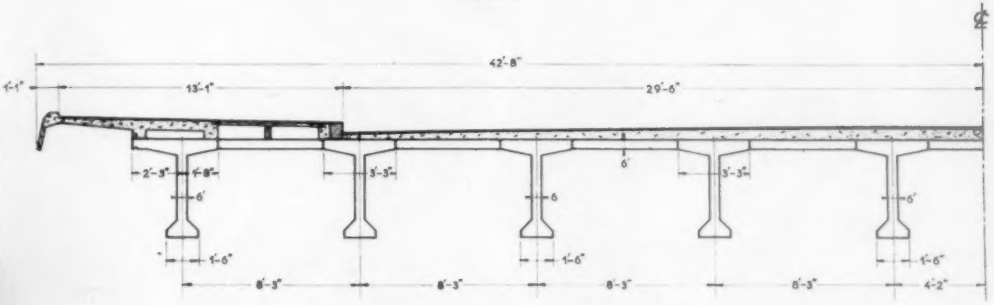
The tubular bridge sections were precast on shore and moved into place on barges before the superstructure of the piers was completed. Each element was then raised with hydraulic jacks, just above its final position, the piers were completed and the girders lowered into place. The whole project involved handling of 25,000 tons of precast members.

Fig. 32



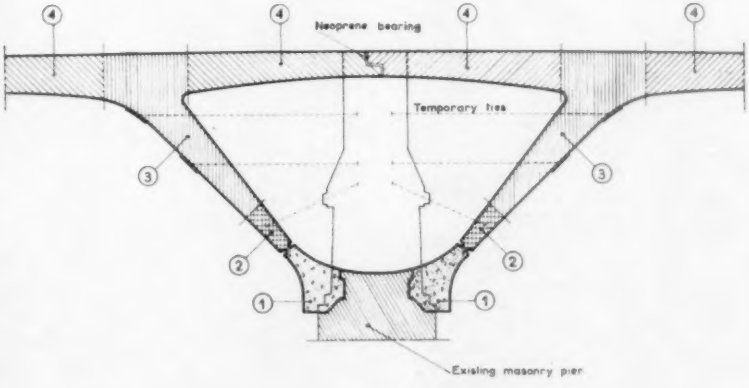
SAINT MICHAEL BRIDGE AT TOULOUSE
PARTIAL LONGITUDINAL SECTION

Fig. 33



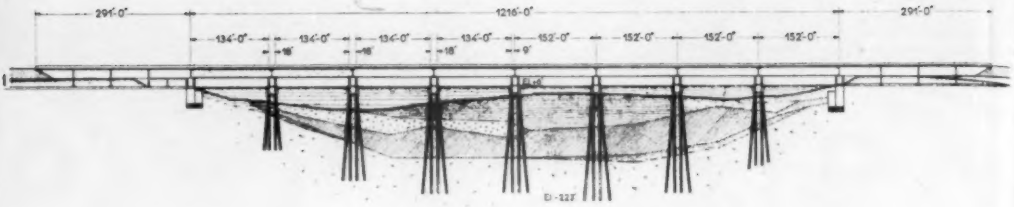
SAINT MICHAEL BRIDGE AT TOULOUSE
TYPICAL SECTION

Fig. 34



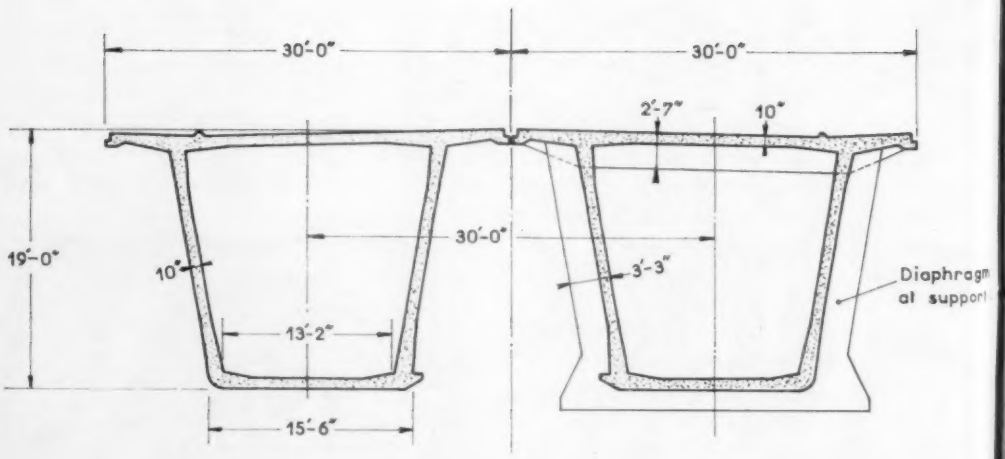
SAINT MICHAEL BRIDGE AT TOULOUSE
CONSTRUCTION SEQUENCE AT TYPICAL PIER

Fig. 35



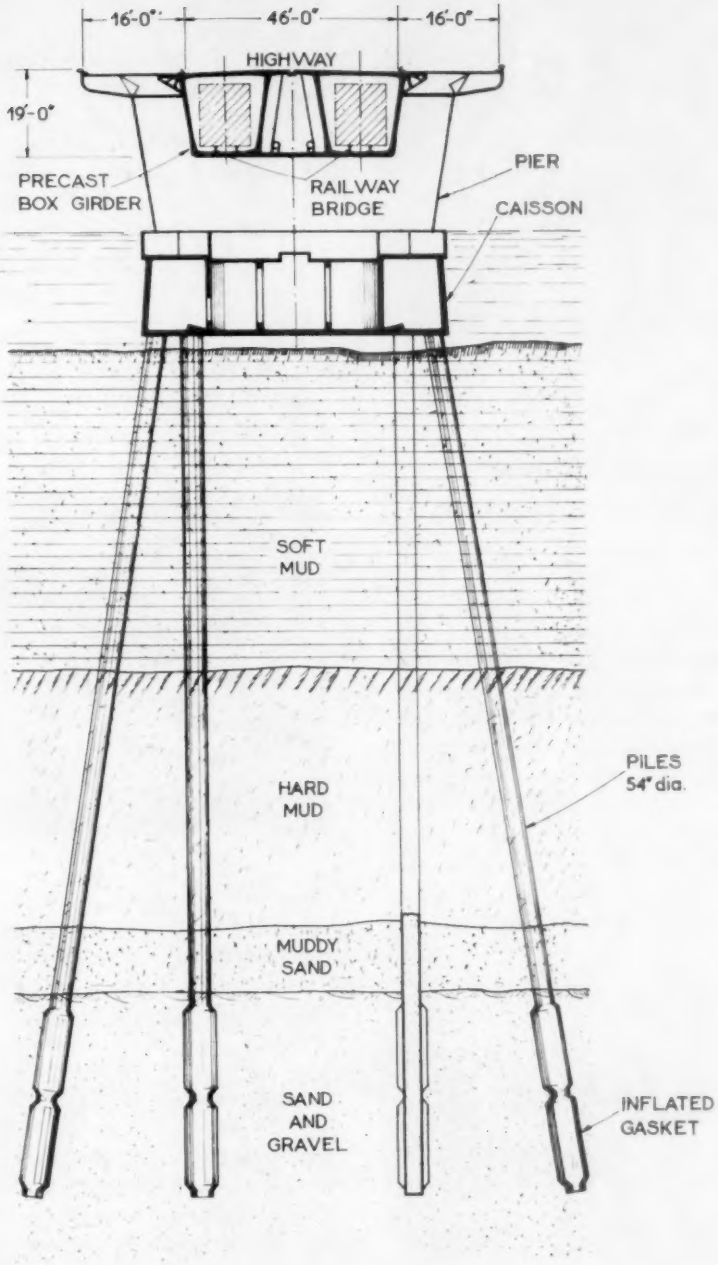
ABIDJAN BRIDGE
LONGITUDINAL SECTION

Fig. 36



ABIDJAN BRIDGE
DETAIL OF PRECAST BOX GIRDERS

Fig. 37



ABIDJAN BRIDGE
TYPICAL CROSS SECTION

Fig. 38

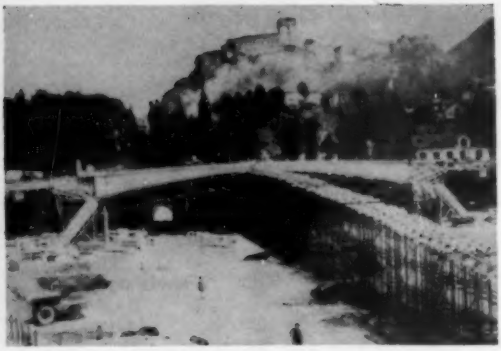


Fig. 39

CHAPTER IV

Unusual Recent Buildings

1)—The Basilica at LOURDES:

This underground church has a covered area of 130,000 sf. shaped as an oval 660 ft by 266 ft. (see fig. 42). The prestressed concrete roof framework consists of 29 double-hinged frames laid out in a fishbone like pattern connected at the crown by a longitudinal diaphragm and near the supports by a peripheral sill. The structure is bounded by a retaining wall located next to a continuous ambulatory 10 ft. wide. The center oval of 600 ft. by 200 ft. located inside the line of the arch hinges is thus free of all intermediate supports.

Each arch consists of two triangular frame legs made up of a strut, a tie, and a curved shallow beam which extends beyond the supports to rest upon the retaining wall. The deck consists of thin arched slabs connecting the upper flanges of the frames. Figures 43 and 44 give the outline of the main structural members of the roof framing.

Another unusual feature of the basilica is that the structure is entirely built upon river deposits with a water table located 10 ft. above the floor low point. The peripheral retaining wall which was designed for hydrostatic pressure consists of a continuous sheet pile curtain extended downwards by a grout cut-off wall. The small residual infiltration flow is permanently collected and pumped out to the river. Figures 39, 40 and 41 represent the construction of the framing cast in place on rolling scaffolding, details of reinforcement and prestressing units for one arch, and an underside view of the finished structure respectively.

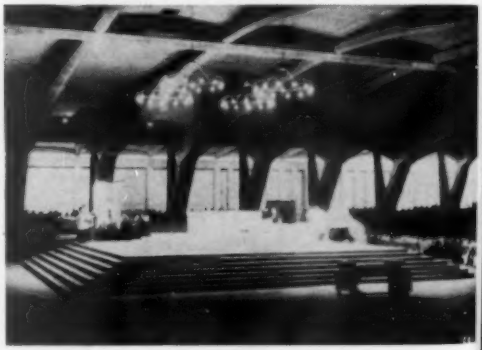
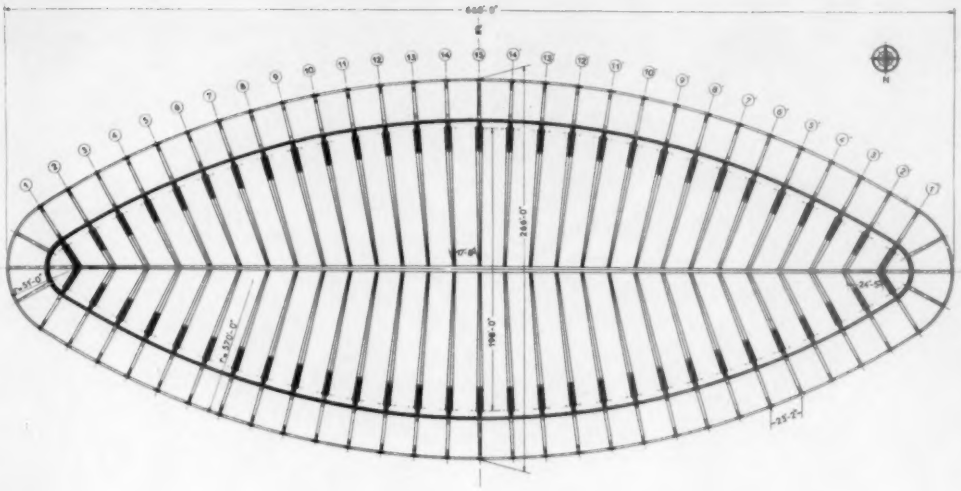


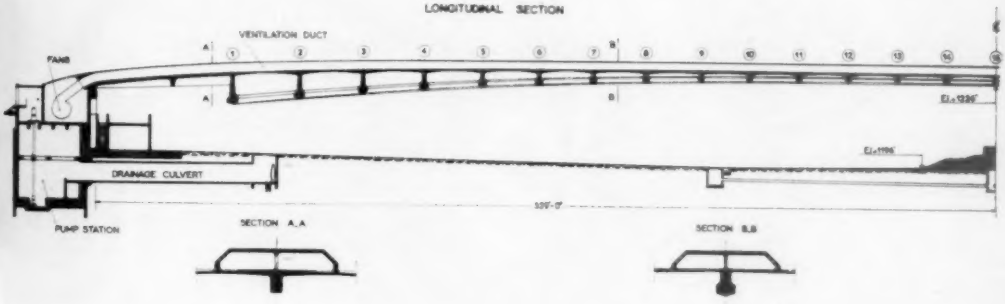
Fig. 41

Fig. 40

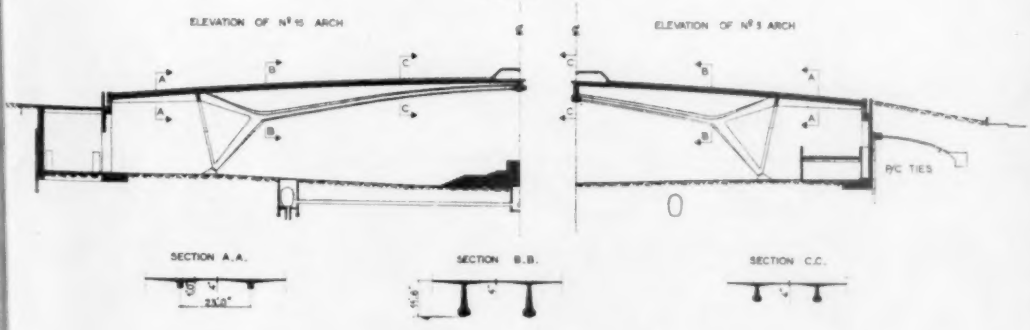
2)—C.N.I.T. Exhibit Hall at PARIS, France:



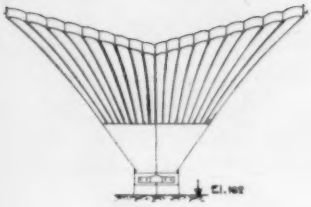
LOURDES BASILICA
PLAN VIEW OF ROOF
Fig. 42



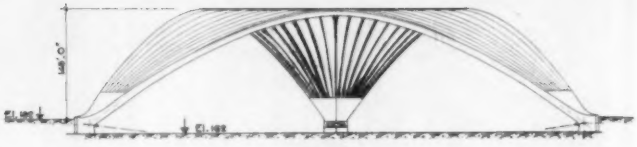
LOURDES BASILICA
CENTER DIAPHRAGM
Fig. 43



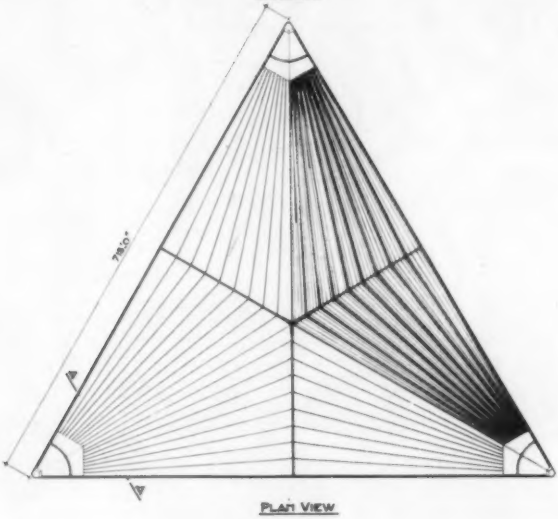
LOURDES BASILICA
TRANSVERSE SECTIONS
Fig. 44



Section A-A



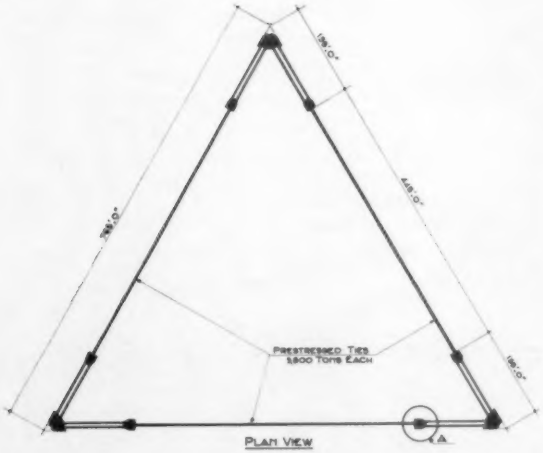
Elevation



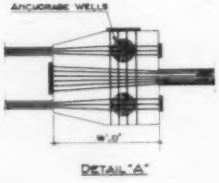
Plan View

C.I.I.T Exhibit Hall
General Layout

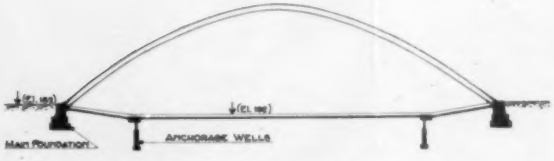
Fig. 45



Plan View



Detail "A"



Elevation

C.I.I.T Exhibit Hall
Foundations of Dome Roof

Fig. 46

This structure is covered by the world's largest dome roof with an area of 240,000 sf. and clear spans of 780 ft. All floors represent a total area of 720,000 sf. made up of factory produced precast units.

a)—Dome roof:

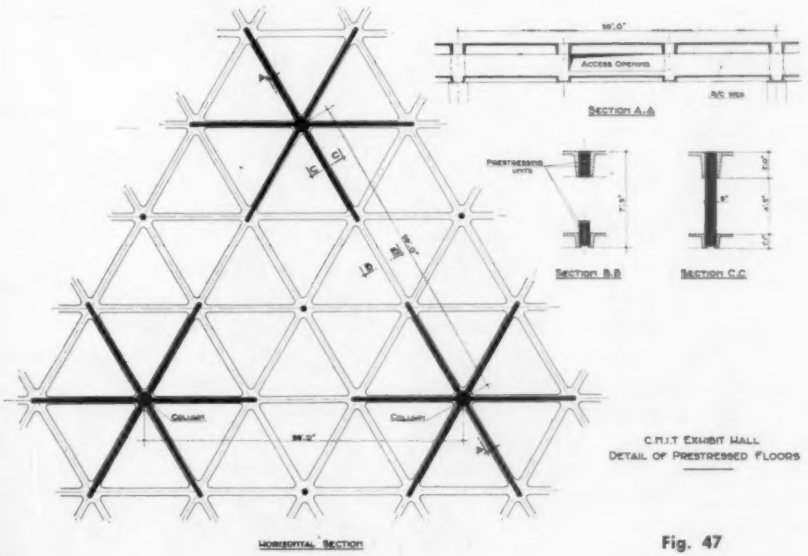
Considerable care was given to the problem of buckling. Figure 45 shows the general layout of the final design adopted for the dome which is shaped in plan view as an equilateral triangle. The dome consists of two slabs (only 2½ in. thick at the crown) doubly curved, stiffened by vertical connecting ribs such as to provide for a box section. All ribs converge towards the three apex's of the triangle where a solid section is used for transfer of the thrust to the foundations. Between the center of the structure and the center of each of the three facia arches, three diaphragms take up the unbalanced thrust induced by the directional change of the intermediate arches at the middle. The average thickness of the roof is less than 7".

b)—Prestressed concrete ties for the dome:

They are designed to resist the 3800 ton component of the thrust of the arches at each apex (see fig. 46). In order to clear the main floor in the center area of the hall, each tie is tapered down at both ends. At the point of directional change, anchorage wells prevent any upward movement of the ties.

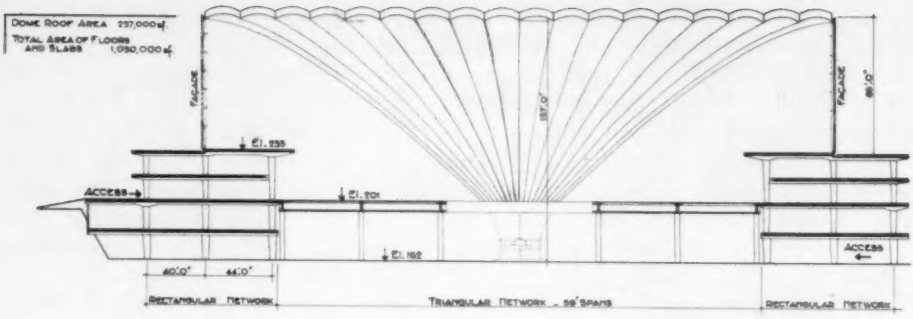
c)—Floors: The center part of the floor at elevation 201 (see fig. 48) is of prestressed concrete construction with an area of 124,000 sf. without expansion joints. The network of beams is laid out in a triangular pattern with a 59 ft. column spacing in any direction. A double concrete deck connected by vertical ribs (provided with access openings) gives a complete service floor beneath the entire exhibit area (see fig. 47).

Placing and assembly by prestress-



C.I.T. Exhibit Hall
Detail of Prestressed Floors

Fig. 47



C.N.I.T. EXHIBIT HALL
TYPICAL SECTION

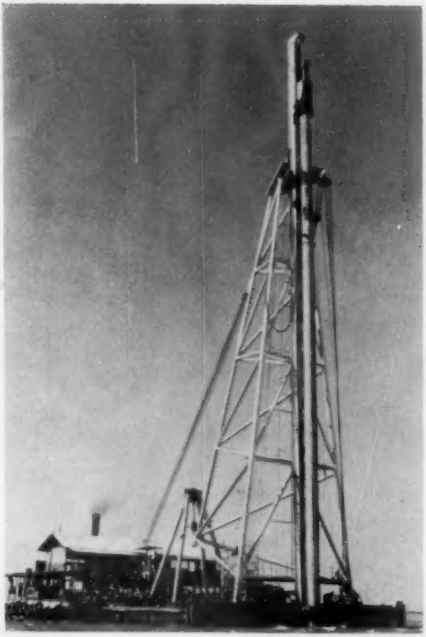
Fig. 48

ing of the entire floor at the job site was done in 7 months.

The author wishes to thank the many organizations connected with the various structures who were gracious enough to furnish photographs and information.

Cylindrical Hollow Prestressed Concrete Piles

By P. S. HEEREMA
Member of the Royal Institute of Engineers—The Hague, Holland



Driving 200 ft. long prestressed concrete pile.

GENERAL

Lake Maracaibo in Western Venezuela covers some of the world's largest oil reservoirs. Drilling is carried out from drilling barges and platforms that support the power and rotary units and derricks. These platforms in water up to 110 ft. depth, are supported by piles. Originally steel piles and reinforced concrete piles were used. The former ones are attacked by corrosion, the latter do not always stand up to the required hard driving and become very heavy for great lengths.

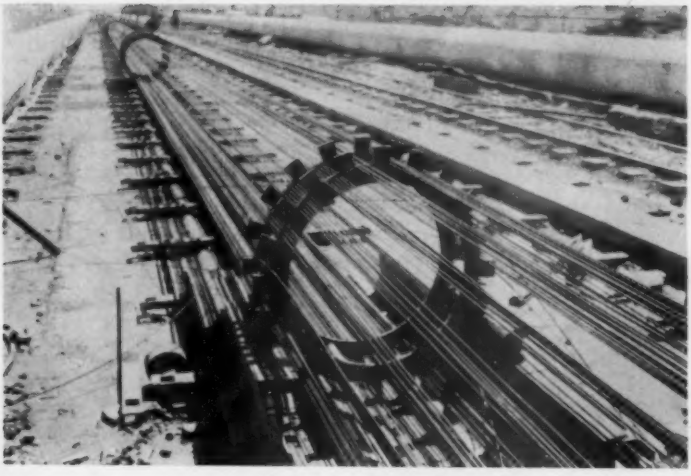
Lately hollow prestressed concrete piles have entered the field. They are a natural for such deep foundations in aggressive water. A drilling platform was designed to be supported by 8 such cylindrical piles. The dimensions of these piles were, an outside diameter of 36 in., and a wall thickness 5 in. Lengths up to 200 ft. were cast in one piece. An ingenious and rapid soil investigation method measures the point and frictional resistance of the soil. This information is relayed by radio to the main office, where a soil mechanics engineer decides on the pile lengths and issues orders to the yard for dispatching the proper piles. Various lengths, ready for use are kept in stock.

FABRICATION

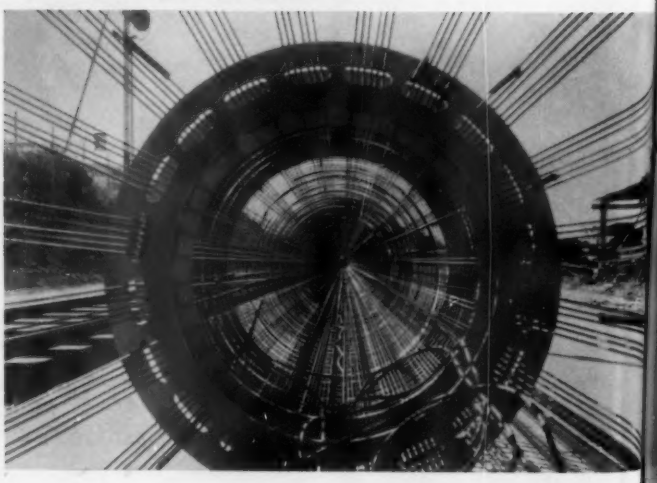
Before deciding on the type of pile, our firm fabricated and tested square, rectangular, hexagonal, octagonal and even H-shaped piles. The merits of post-tensioning and pretensioning systems were carefully evaluated. We finally decided on the pretensioned monolithically cast, cylindrical hollow pile with a 36 in. O. D.

In order to produce a pile of this length with uniform density and concrete strength, extensive testing and research was necessary. This testing included the location of exterior vibrators determining their direction of impulse waves plus designing the mix and the pouring

General view of casting beds.



Prestressing the wires.

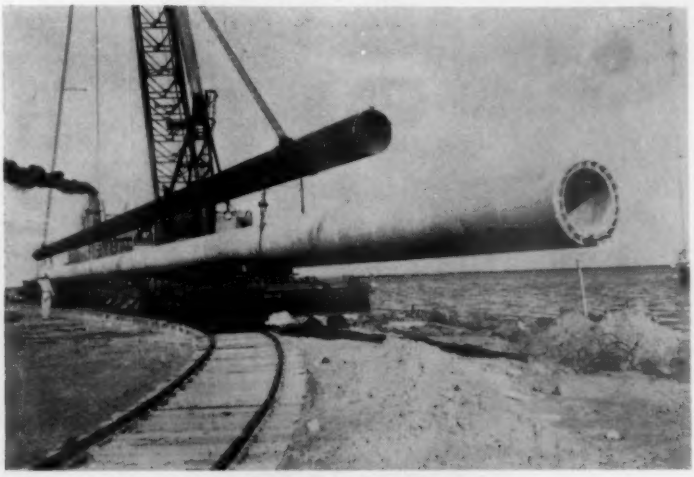


Wires prestressed, stirrups placed; note some field splices in the wires.

procedure. Numerous tests resulted in a mix that satisfied requirements for strength and durability. The mix contained 8½ bags of cement per cubic yard and had a slump of ¾ in. The relation between the quantities of sand and stone was 1 to 6.5, pozzolith No. 8 was also added for density. With natural curing this mix reaches a strength of 4,000 psi in 32 hours, after which the prestress force is transferred to the concrete and the piles are removed from the casting bed. At 7 days the concrete strength averages 6,-800 psi and at 28 days, 8,000 psi. The strength tests were conducted on standard 6 in. by 12 in. cylinders.

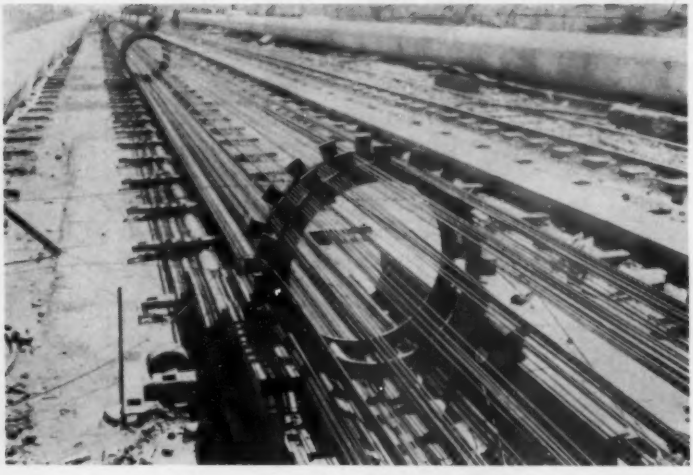
On casting beds, over 800 ft. long, high tensile strength steel wires were stressed in one operation by pulling an end bulkhead with two 200 ton hydraulic jacks. The wires are of heat treated alloy steel, with oval cross section and ribs to improve bond. These wires can be

Concreting operations; note the exterior vibrators.

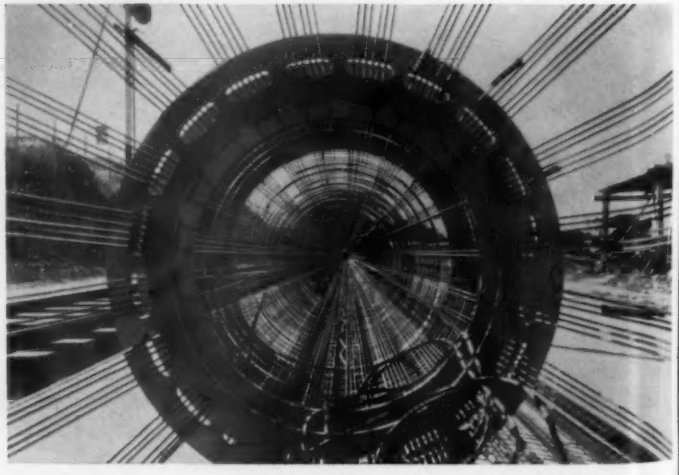


The revolving 60—T crane lifts a pile.

General view of casting beds.



Prestressing the wires.



Wires prestressed, stirrups placed; note some field splices in the wires.

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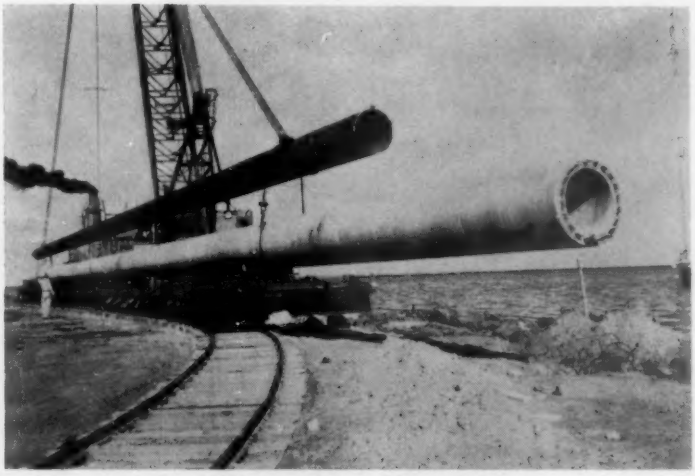
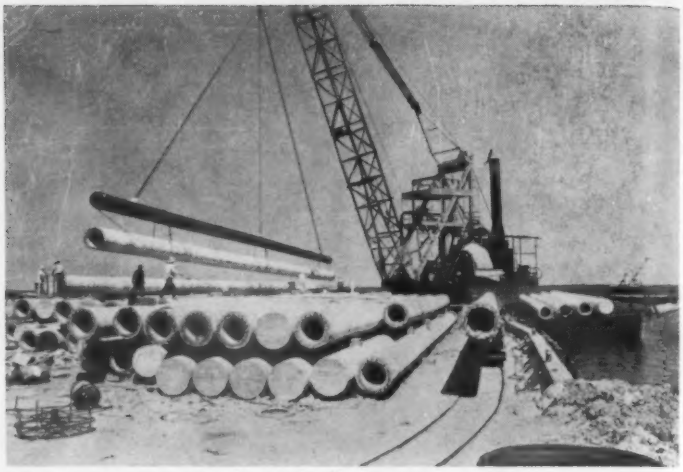
Concreting operations; note the exterior vibrators.

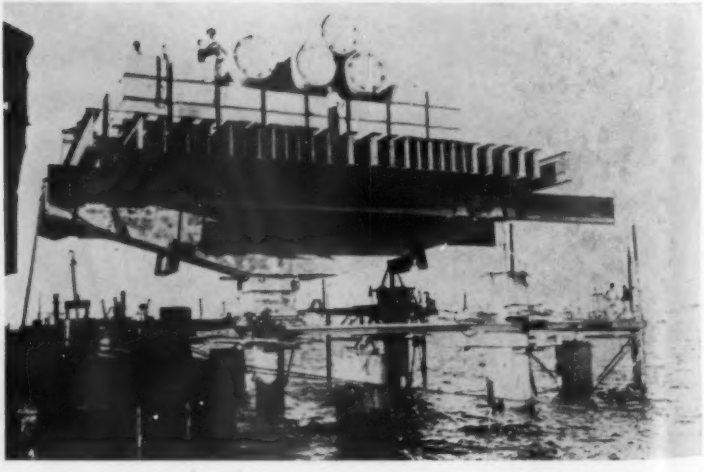


The revolving 60—T crane lifts a pile.

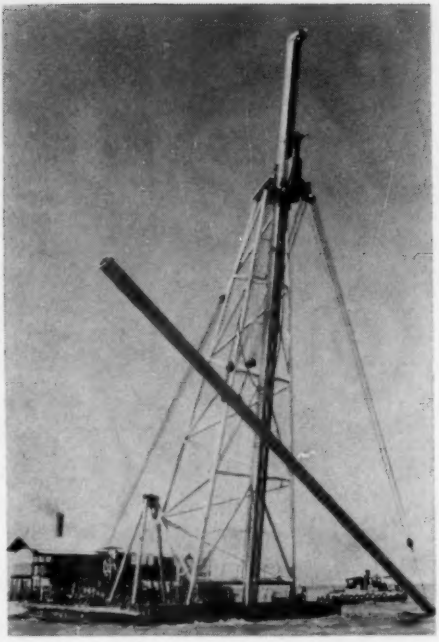


The pile being placed on the stockpile; note some flat points.

The bending test on a 45 foot section of a 36 in. pile.



A 600 Ton load test.



The floating piledriver hoists a 195 ft. pile on a 2-point lift.

spliced with a Voigt machine (B.B. R.V. patent). The number of wires used per pile depends on the required prestress force, which is mainly governed by the bending moments during handling. The stress is kept at 10% over the initial design stress of 118,000 psi for 10 minutes. The yield/break-strength is 206,000/226,000 psi.

Mild steel spiral stirrups are then placed, followed by steel outside forms and an inflated rubber tube, reinforced by steel sleeves, forms the hollow circular void. Exterior vibrators are placed at staggered 5 foot intervals. This vibration combined with the compaction of the concrete produces a very dense concrete in the bottom and sides of the pile. The top, however, needs additional vibration, which is accomplished by needle vibrators and by finishing with a floating vibrator. Test cores taken from all parts of the pile section show little spread, and average 23% higher than the test cylinder.

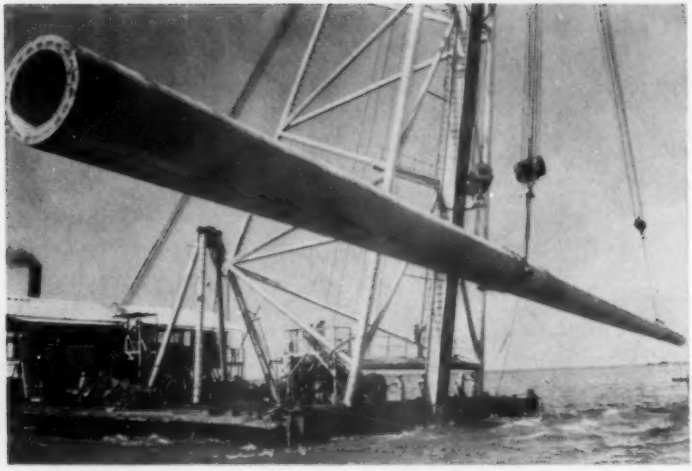
HANDLING AND DRIVING

Piles are removed from the casting bed by an overhead gantry crane with a 4 point pickup. They are then placed on railroad cars and transported to a 60 ton revolving crane that loads them on a stockpile or on a barge. They next proceed to the floating piledriver that hoists them using a 2-point lift.

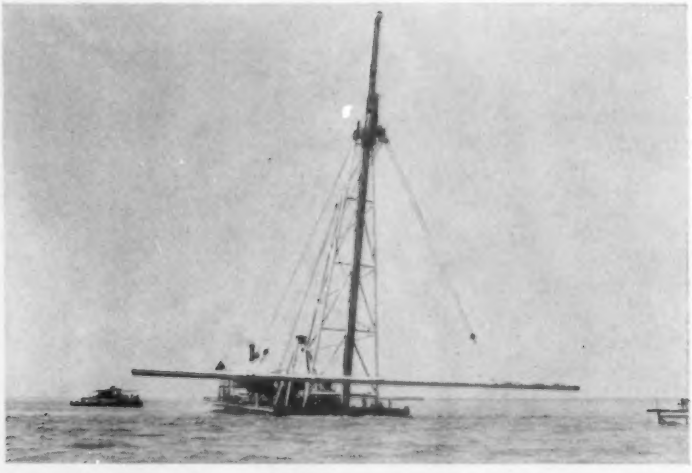
We use heavy single acting steam-hammers, with either a 16.5 or a 22 ton ram, the stroke of which can be manually controlled between 6 in. and 4 ft. We feel that the relation between weight of ram and weight of pile should be at least 0.25. The higher this relation is, the better, as the pile can be driven to greater bearing capacity with less damage.

After using a 16.5 ton ram for years, we recently acquired a steam-

The Menck steamhammer with 16.5 T ram driving a pile.



200 ft. pile being moved to position.



200 ft. prestressed pile.

hammer with a 22 ton ram. It was especially designed and built for us by Messrs. Menck and Hambrock of Hamburg, Germany. This hammer is believed to be the heaviest and most powerful hammer in the world. It can deliver a 176,000 ft. lb. blow at a 4 ft. stroke. It will be used on foundation work for the bridge over Lake Maracaibo where extremely hard driving is expected.

Initially we drove the piles open-ended but abandoned this practice when it was found that longitudinal cracks appeared occasionally; we think this is caused by water hammer and internal pressure. Since then we drive closed-end, empty piles, and longitudinal cracks have not appeared any more.

Occasionally fine horizontal cracks have been detected in piles with soft driving. We believe that excessive tension during soft driving with little point resistance, but appreciable shaft friction, is mainly responsible for such cracks. These cracks do not go through the wall and do not admit water. Divers report the presence of exudate, and we believe they heal themselves. We take two steps to remedy this situation. First the stroke of the ram is controlled by hand and constantly adjusted to driving conditions. Short strokes for soft driving and long strokes for hard driving. Second the design tension in the steel is kept so low that at the moment of driving it is below 103,000 psi or 50% of the yield limit. The steel is then still able to absorb the large tension waves caused by driving, before the yield limit is reached, at which point horizontal cracks might appear. With these precautions we have driven piles continuously for over three hours with the 15 T ram and a 39 in. stroke to a final resistance of 150 blows per foot, without damage to the pile head or shaft.

TESTS

Impermeability tests on a pile section with an internal water pressure of 170 psi left the concrete on the outside face dry. A bending test was also performed on a 45 foot section. The pile failed at 1600 foot kips. The first cracks appeared at 910 foot kips. After unloading the pile the cracks closed at 660 foot kips. Even when the pile was unloaded from 96% of the failure load, the cracks still disappeared. The concrete cylinder strength at the time of the bending test was 6500 psi. With our present improved mix of 8000 psi the bending test would give even better results.

Various pile load tests have been made periodically. One pile was loaded to 600 tons and had a gross settlement including elastic shortening, of 1 inch. The net settlement after the load was removed was ½ in. Another pile loaded to 470 tons had a gross settlement of ⅜ in. and a net settlement of 1/16 in.

CONCLUSION

With strict control of all phases of the fabrication, handling, and driving operations; prestressed concrete piles of up to 200 ft. length can be used successfully for great bearing capacities.

2 and 3-Wire Strand For Prestressed Concrete

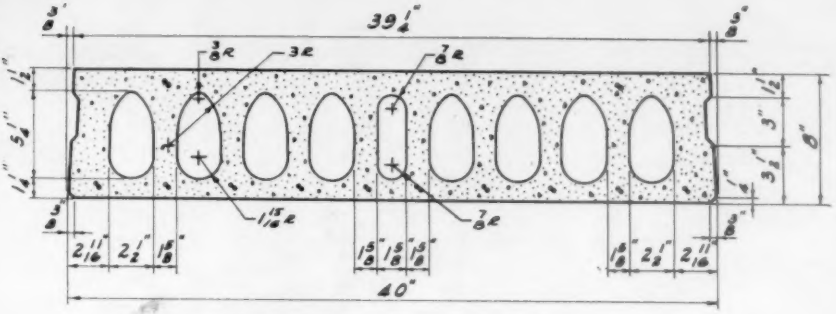
By: R. O. Kasten

Director of Research, Union Wire Rope
Kansas City, Missouri

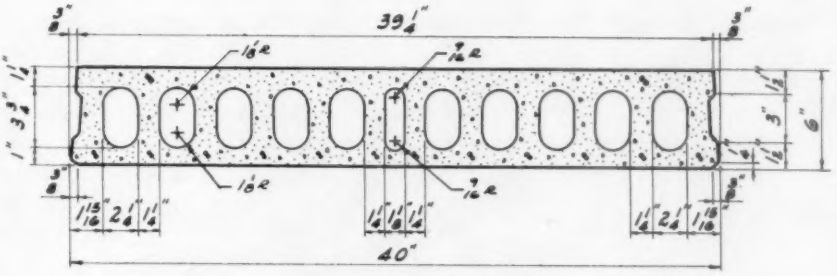
7-wire strand, starting at size 1/4 in. (9,000 lbs. minimum catalog strength) up to 1/2 in. (36,000 lbs. minimum catalog strength), has been manufactured by most strand producers. For bridge beams and long double tee spans, 3/8 in., 7/16 in., and 1/2 in. sizes have been the most popular. While during the early years of prestressed concrete considerable attention was focused on bridges, producers are now re-evaluating their plant layouts for more efficient material handling. They are looking for a wider variety of products to keep their plants operating continuously in order to have a respectable balance sheet at the end of the fiscal year.

Material handling is a costly item and must be classed as a direct cost in the manufacture of prestressed concrete. Naturally, we immediately think more about assembly-line manufacturing practices, the very miracle that made America great by giving us more and better things or, perhaps we should say, raising our standard of living. The automotive industry is typical of what assembly-line production methods can do. Now, how can such methods be incorporated into the manufacturing of prestressed concrete?

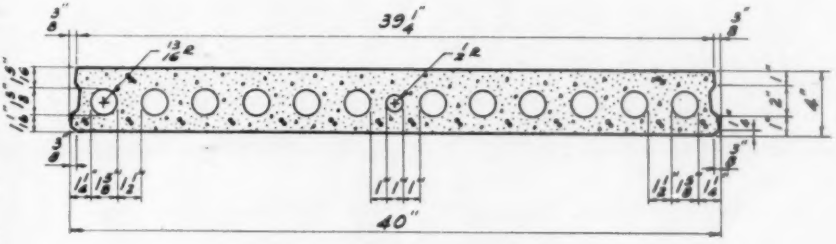
Hollow core, machine extruded, flat slabs are being and have been manufactured for over five (5) years on nearly an assembly-line or automated method with extrusion machines. In Figure 1 are shown cross-sections of 4 in., 6 in., and 8 in. hollow core, flat slabs. For uniform prestressing and camber control, a steel tendon or strand is required between each void space as one can see in Figure 1. In these slabs, between ten (10) and fourteen (14) strands are required for pre-tensioning. The load the strands must carry, naturally, will vary according to the span and depth of the slab. It doesn't take one long to make a few elementary stress analysis calculations and learn that the present 7-wire strand tensile strength is far above the necessary requirements for hollow core flat slabs. Consequently, a new standard is being developed whereby tensioning elements either in the 2 or 3-wire strand are available to the prestressed industry. It has been the policy of most strand producers to work closely with the prestressed concrete producers and to fulfill their needs as economically as possible, that is, give them a tensioning element that does exactly the job and not try to furnish a strand larger than necessary. With this policy, the 2 and 3-wire strand, shown in Table 1, has been developed. Here are shown the sizes, description, minimum tensile strength, approximate area and weight per 1000 ft. The yield strength and ultimate elongation



8" SPANCRETE



6" SPANCRETE



4" SPANCRETE

Figure 1

Courtesy of West Allis Concrete Products Company, Milwaukee, Wisconsin.

Description and Nominal Diameter	Weight Per 1000′ in Pounds	Approx. Area Sq. In.	Minimum Strength Pounds
2-Wire 13 Ga. (.092) 3/16″ Dia.	47	.0133	3,500
2-Wire 12 Ga. (.105) 7/32″ Dia.	61	.0173	4,500
2-Wire 11 Ga. (.120) 1/4″ Dia.	79	.0226	5,900
3-Wire 13 Ga. (.092) 3/16″ Dia.	71	.0199	5,200
3-Wire 12 Ga. (.105) 7/32″ Dia.	92	.0260	6,700
3-Wire 11 Ga. (.120) 1/4″ Dia.	118	.0339	8,700

Table 1

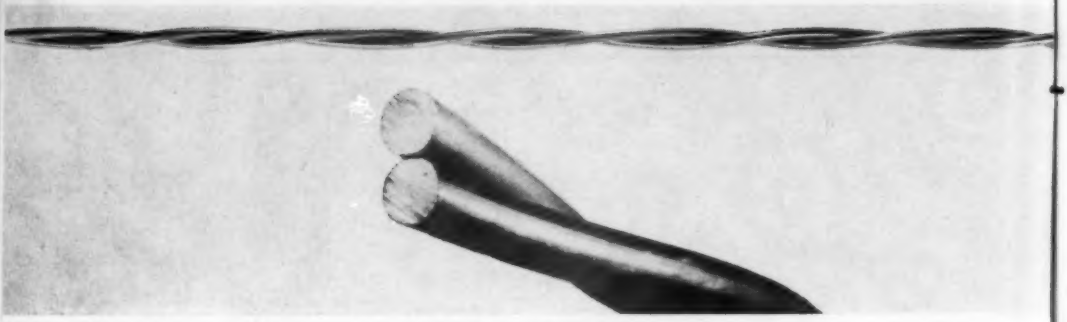


Figure 2—2 wire strand.

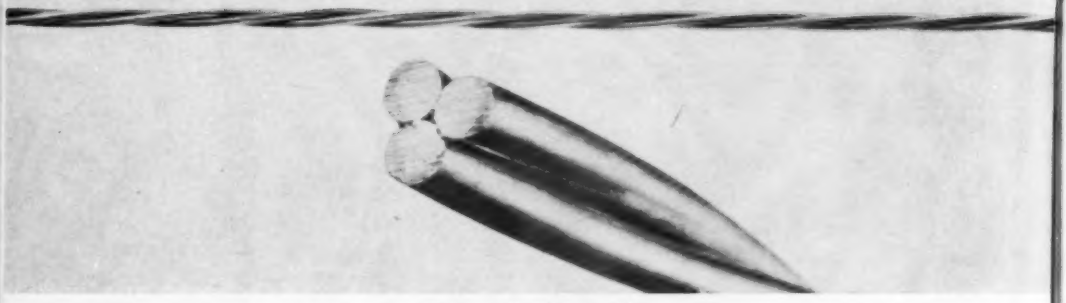


Figure 3—3 wire strand.

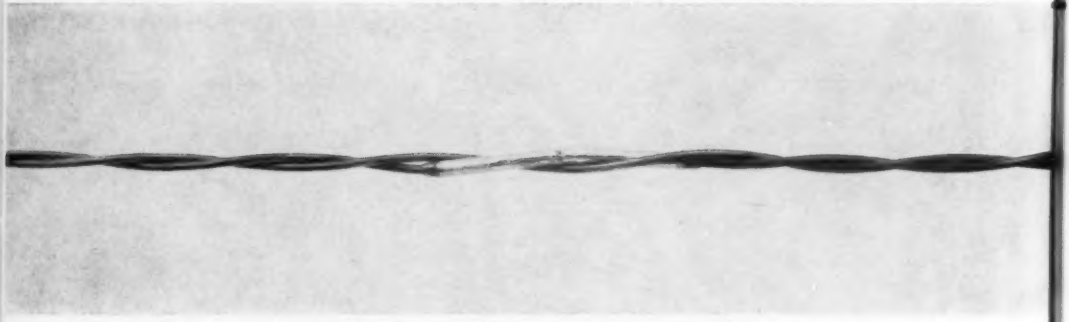


Figure 4—Tapered sleeve joint in 2-wire strand.

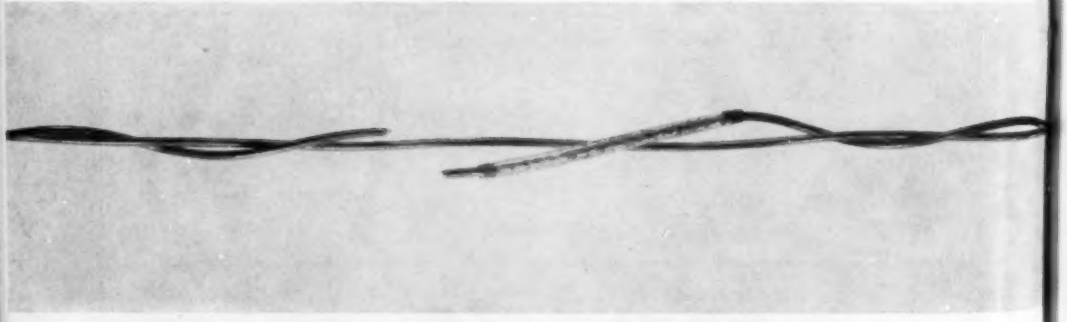


Figure 5—Length of 2-wire .240" strand with joint.
This joint develops the minimum catalog strength shown in Table 1.

will be approximately the same as shown in ASTM A 416-57T. Fig. 2 and 3 are close-up pictures showing the 2 and 3-wire strand respectively.

With longer pretensioning beds and more nearly automated operations, prestressed concrete producers are requiring longer continuous lengths of strand for efficient operation. This presents a far more serious problem with 2 and 3-wire strand than with 7-wire strand. With 7-wire strand, closely controlled electric resistance butt welds are used very satisfactorily. However, in the 2 and 3-wire strand, welds are not usually permitted unless the strand develops the minimum strength requirements. That made necessary the development of a tapered sleeve joint which will develop 100% of the minimum strength shown in Table 1 at the sleeved joint. Fig. 4 shows a tapered sleeve joint in a 2-wire strand, while Fig. 5 is a close-up view of a sample of 2-wire .240" strand which failed in tension at 6,010 lbs., over 100 lbs. above the minimum strength. Also, you will be interested to know that the sample failed away from the joint. Almost any length of strand can be furnished with the limiting factors being the reel size and the equipment available at the prestressing yard.

In Table 1 are shown three (3) sizes of 3-wire strand giving minimum strengths from 3,500 lbs. to 8,700 lbs. to meet the customer's specific needs. Actually, 2 and 3-wire strand can be made in any size wire to meet the specific customer requirements. Naturally, the volume must be sufficient to justify a production run.

With the strand producer and the prestressed concrete manufacturer working together, prestressed concrete will become a more economical and competitive product.

Neat Joints Are Good Business

by **Laurence Cazaly***

Presented at the Prestressed Concrete Institute Convention
New York City, September 28, 1960

Todays architecture places great importance on the structure. To appear in a magazine a building must have a structural frame like a flower arrangement topped with well wrinkled prunes or it must be an endless repetition of the latest magic module. Fortunately we have many sources of theory to analyse the first, and manufacturers have made tremendous advances in cost, production and quality control, to remain competitive in the second. Nevertheless, if designers are going to meet their estimates and contractors their costs, they must recognize that half an arch improperly lifted may have higher stresses under its own weight than the complete structure under live load. Also that while 5 tons of concrete may leave the plant costing less than 1 ton of steel, wood, plastic or aluminum, from then on all the advantages are with the lighter material. Joint and erection details are not for the junior draftsman. They should be given painstaking consideration by the top echelon of both designer and manufacturer. If a joint can be fabricated by a welder just going on vacation, placed in a mould by a carpenter with a linen tape, erected with the only piece of equipment within 50 miles on a rainy day by a man who has just quarrelled with his wife, and if the engineer pronounces it safe, the architect says it is beautiful and it did not cost you more than $5.00, rest assured that the design was a good one.

*Consulting Engineer
Toronto, Canada

Some Elementary Rules of Jointing

Joints must:—

a) Withstand building movements without breaking down.

b) Absorb concentrations of stress and strain.

c) Occupy the minimum space and present a neat appearance when exposed.

For economic reasons they must be:—

d) Safely formed by normal labour with no allowance for "off days."

e) Cheap to fabricate without excessive carving of formwork.

f) Capable of erection in all weather.

g) Fast to erect without cranes and other trades.

h) Able to take up a considerable amount of tolerance.

All joints should meet this check list. One family will be considered here.

Joints of fabricated structural steel.

Structural steel can be highly stressed in any direction, is conveniently ductile, and fabrication techniques are well understood. Properly used it can be a rugged and invisible connecting link between concrete parts. Too often this link is simply a weld plate and anchor bar or a many-splendoured thing of bolts and welds. In designing

FIG. 1

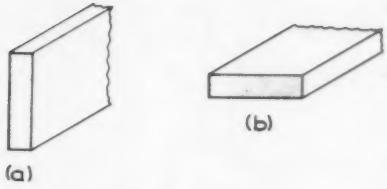


FIG. 2

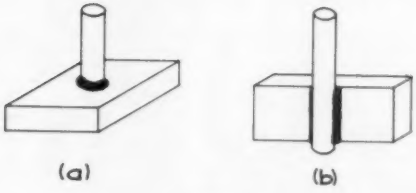


FIG. 3

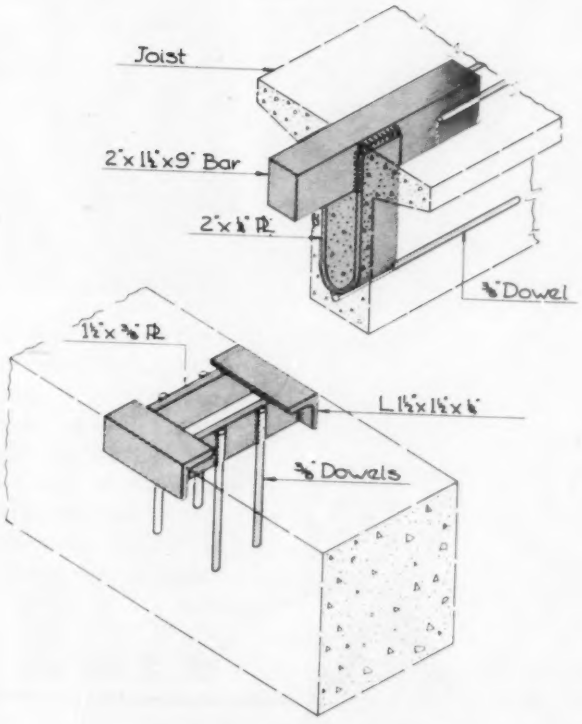
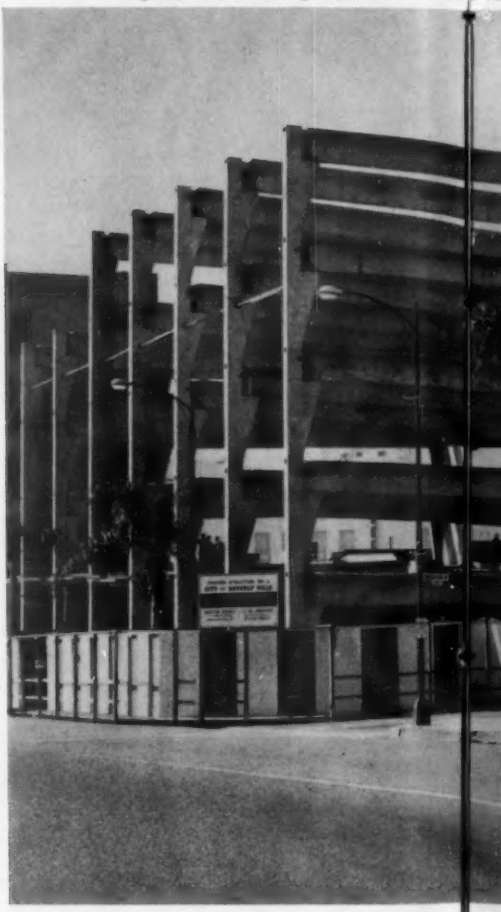


FIG. 4

PRINT IN BINDING

Owner: City of Beverly Hills, Cal.; Architect: Walton
Angeles, Cal.; Consulting Engineers: T. Y. Lin & As



PLASTIMENT increases workability of lightweight concrete

The five-story Beverly Hills Garage illustrated above, providing parking space for 400 cars, is a unique precast, prestressed structure. Long spans of 75 feet eliminated columns in parking areas and use of lightweight aggregate minimized horizontal and vertical loading.

Use of Plastiment Retarding Densifier increased the workability of the Ridge-

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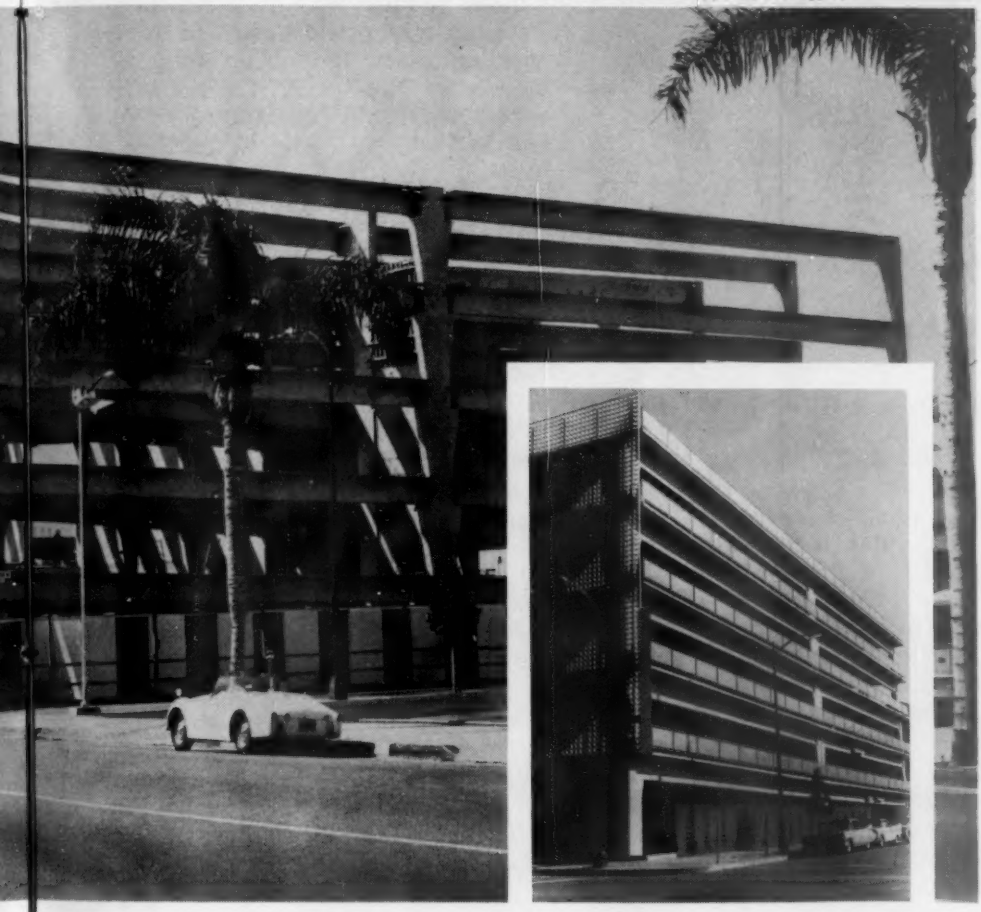
SIKA CHEM

: Walton Becket & Associates, Los Lin & Associates, Van Nuys, Cal.; **General Contractors:** C. L. Peck, Los Angeles, Cal.; Ellis E. White Co., Los Angeles, Cal.; **Precast Columns & T's:** Wailes Precast Concrete Corp., Sun Valley, Cal.



lite lightweight concrete thereby assuring a smooth, clean appearance to the exposed concrete members. Using high early cement, strengths averaged over 3,500 psi in 16 hours in the precast elements. Slab concrete placed at the site reached 3,500 psi in 2 days. 28-day strengths averaged 6,500 psi.

Mix contained 7-1/2 bags of cement per cubic yard and 2 fluid ounces of Plastiment per sack of cement.

Plastiment features are detailed in Bulletin PCD-59. Ask for your copy. District offices and dealers in principal cities: affiliate manufacturing companies around the world. In Canada, Sika Chemical of Canada, Ltd.; in Latin America, Sika Panama, S. A.

EMICAL CORPORATION

Passaic, N. J.

steel joints start from first principles:—

i) Solid bar Fig. 1 (a) is good for shear and torque.

ii) I Sections (a) and (b) in Fig. 2 have some shear and torsional strength but (a) is much stronger in bending in a vertical plane.

iii) Theoretically (a) and (b) in Fig. 3 can both develop full strength of the bar. However (b) is still strong when badly welded because of the large weld area possible, while (a) is often pulled off easily.

iv) Bolts with double nuts provide tolerance in one direction. Bolts with washers in oversize holes provide tolerance in two directions but may slide unless torqued tight or welded. A combination of these two provides tolerance in three directions such as in a column base but such a combination will only resist a large force along the axis of the bolt.

Welds can easily take up tolerance in all directions but if the welder waits for the crane and the crane waits for the welder someone is losing money—fast. In very cold weather welding is difficult or impossible. Welds leave scorch marks on concrete and require exposed steel plates which may lead to exposed rust stains. This is, therefore, a versatile tool to be treated with caution and it is wise to design other temporary connections and use welding to sew the job up tight. **Important Note—ALL GENERAL CONTRACTORS NEED PLENTY OF TOLERANCE** even when they will not admit it.

v) Labour costs in N. America are high but do not assume that material is free. There is always a crossover point when three small fabricated parts cost less than one chunk of material. Until experience makes you sure, it is well to look for both alternatives.

vi) Steel parts in concrete can work in two ways. They either take a load directly and distribute it to the concrete as in a prestressing anchorage and plate, or they strengthen the concrete sufficiently for it to take the load directly as in the reinforcement round an anchorage cone. The right choice can result in great savings in a frequently repeated detail.

These principles are not new, rather, they are so old that they are often forgotten.

The joints described below are all obvious extensions of simple ideas

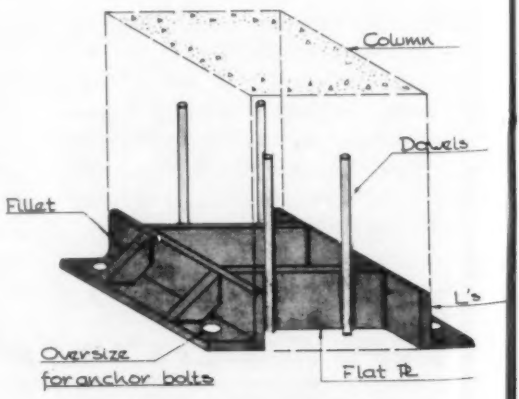


FIG. 5

and yet they are far from being common practice.

Typical Examples of Steel Connections

Joist Hanger

This has two parts, a hanger in the single tie and a seat in the top of the supporting beam. Two hangers and a seat cost $4.00 to $10.00 depending on size of joist and load to be carried. The one shown here (Fig. 4) will carry about 8 Kips or an 18 in. single tee with 500 lb/ft. over a 32 ft. span. The hanger suspends the unit like an open web steel joist. It also provides end and shear reinforcement and in light single tees no other reinforcing bars are used at all. It does not break into a length of standard mould. Welded to the beam seat it provides lateral support. The beam seat binds the edge of the main beam to take the concentrated load from the hanger. It makes a weld plate for the hanger and again does not break up a standard mould since it fits flush. The combination can be recessed slightly for flush tops to beams and joists if required.

In the field the combination erects with ease. The joist is swung into place and not one unit has been rejected or required adjustment in the many hundreds that have been used. (See Picture Page 56)

Fabricated Base plate

This plate (Fig. 5) replaces a solid 1-1/2 in. base plate generally used. The dowels are welded to vertical flats. The vertical flats distribute the loads to wide spread bolts. Angles distribute in a complementary direction. This unit is much lighter and stiffer than a solid plate and the material saving pays for fabrication. Lighter weight makes for easy fixing in the mould.

Curtain Wall fixing

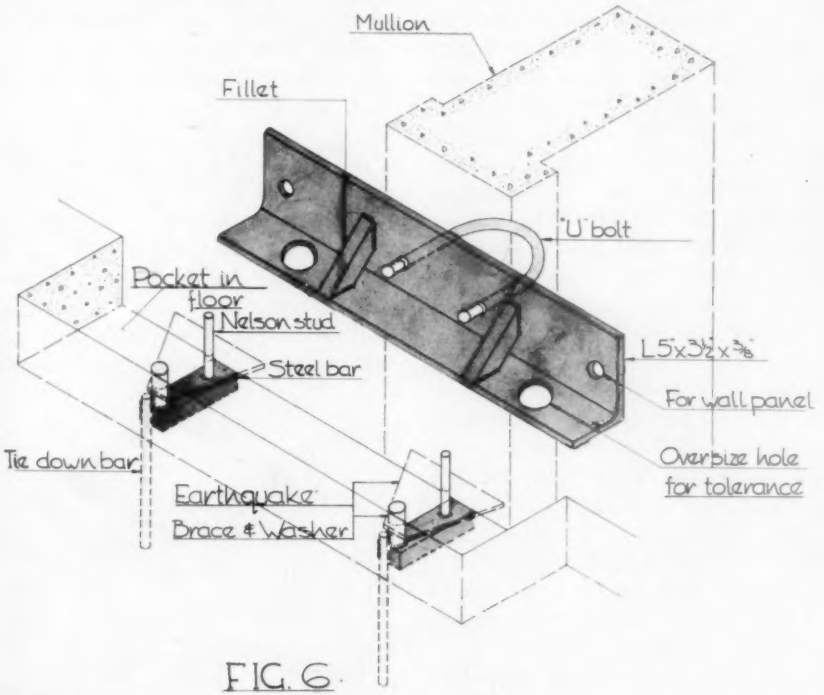


FIG. 6

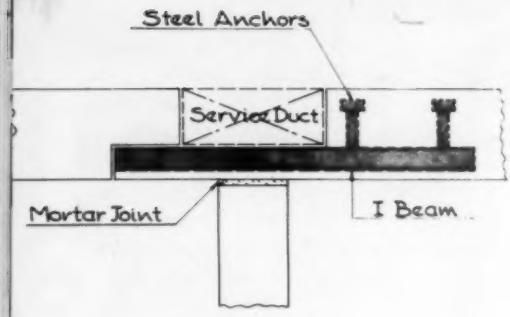


FIG. 7

This is the principle connection (Fig. 6) on the 43 storey Bank of Commerce Building in Montreal. The solid bar seat—like a joist hanger upside down—allows pockets to be made with open tops. In front of columns, space is so restricted that an angle in the edge of the floor would block concrete placing if a pocket were not used. Threaded Nelson studs which locate the angle can also be used by the general contractor to locate the assembly in the floor. The angle which is bolted to the mullion on the ground supports both mullion and wall panels. It takes all tolerance and after the mullions are aligned with a jig, the wall panels must fit. The tie down bar makes the solid bar capable of transmitting the weight of the wall back from the extreme edge of the floor slab. By putting the Nelson stud on the end the mullion angle is reduced to a minimum. 1-1/2 in. tolerance in all three directions is allowed and yet the angle is only 5″ x 3-1/2″. Note that if the fillets were removed the angle thickness would increase, the outstanding leg would be reduced and a 6″ x 4″ would be required instead of 5″ x 3-1/2″. The increase of both thickness and leg size costs more than the fillet. In addition, some of the joints are so tight that the panel would have to be recessed to accommodate the angle between the panel and the column. As it is, all joints, whether in front of columns or ducts, and no matter what the size of column or duct, require the same connection. The resulting simplicity in site administration is worth working for.

An unusual Beam connection

Mechanical engineers have a habit of buffaloing their way through the best of structures. In this case (Fig. 7) a large duct, two thirds of the depth of the beam, was required immediately over the column. A normal half joint in the beam was used on one side but the other contained a structural steel bracket buried in the concrete.

A reinforced Column head

The addition of a light frame (Fig. 8) protects the edges of a column head and prevents spalling when bearing pressures are high. It is cheaper and more reliable than a heavy bearing plate.

It should be noted that in all these examples welding is used either before fabrication or after erection is completed. Preferably the crane can put the piece down with adjustment by a follow up crew. Grouting is also avoided in highly repeated joints so that no change in technique is required from win-

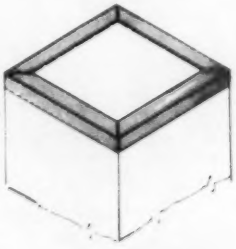


FIG. 8

ter to summer construction.

Conclusion

Design, as opposed to calculation, does not lend itself to rational analysis. It is always subject to the emotional selection of the designer. Two factors omitted from this study are the demands of local climate and the preferences of local manufacturers for a certain technique. Nevertheless, a search for the simplest connection with the fewest parts and for the lightest connection with the smallest amount of material should yield a surprisingly economical blend of detail. 10% to 20% savings on job costs can be made by good jointing. There is no phase in engineering where so great a reward can be obtained by so little effort.

"Prestressing and Its Function in Architecture"

MR. A. KENNERLY*

Presented at the Prestressed Concrete Institute Convention
New York City, September 28, 1960

As an architect in the midst of so many distinguished engineers, I hasten to assure you that my non-technical remarks are designed to encourage your group to provide us architects with ammunition—the ammunition we require to exploit to the ultimate this fantastic and potentially beautiful structural medium you call prestressed concrete.

I firmly believe prestressing to be the most advanced construction expression of this century. Therefore, I offer these frank and informal observations because I believe them to be to our best mutual interests.

Architects generally are a progressive breed. They welcome new ideas, products and methods, especially those which contribute excitement and imagination to the solutions of their clients' problems and needs. But, at the same time, the experienced architect is cautious in his approach to the unorthodox, and wants to know the answers to many questions before his full support can be enlisted.

Apropos of this—now, I've talked to many architects and engineers, and, to a man, they are enthusiastic about the intriguing possibilities inherent in shallow depth long span construction. But their enthusiasm sometimes begins to fade if annoying details begin to make it difficult to incorporate prestressing in the project at hand, whereupon they take refuge in the relative security of steel and conventional concrete as time-tested structural expressions.

As a matter of fact, until comparatively recently, most architects' awareness of prestressed concrete have been limited to its pure engineering aspects and its contributions to every construction.

The sensational results accomplished in this field have been so well publicized that almost everyone is familiar with the sleek long span bridges that have become a part of our super-highway program, and has learned to associate prestressing with water towers, stadium and convention halls. Yet I daresay that many of us do not realize that prestressing has already become a function of architecture in most parts of this country and has been used in a wide variety of building types from Maine to California.

To illustrate this point, I should like to mention a few examples:

In Seattle, Washington, a $12 million, 21-story office building in prestressed concrete, and a $3 million high school;

In Cambridge, Massachusetts, a multi-story laboratory building.

In Honolulu, Hawaii, a $40 million shopping center and office building complex;

In Hempstead, Long Island, a multi-tier parking garage for Abraham & Straus;

In Fort Lauderdale, Florida, a church and parish hall;

In Alpena, Michigan, a Kroger Supermarket;

* Partner
Skidmore, Owings and Merrill
New York City

Near Chicago, Illinois, an administration building and training center for United Airlines;

In Springfield, Illinois, a high school auditorium, in which, incidentally, the engineers and construction forces utilized the prestressing jacks as hoists to load the huge post-tensioned girders in place;

In Lister Hill, Alabama, a large basic aluminum plant for the Reynolds Metals Company;

On the shores of New York's Hudson River, a vast new steamship pier, a terminal designed for the handling of both passengers and freight. This, however, is an exceptional case in this town of ours because the Department of Marine and Aviation, and not the Building Department, controls construction activities on the waterfront. We'll touch on this a little later.

And in Los Angeles, even residential construction is involved.

These examples of your industry's participation in building construction are indicative of the ever-increasing role prestressing will have in architecture. They suggest that in a short time prestressing's enormous contributions will bring it abreast of the conventional structure systems with which we have been working for so many decades. Also, it is gratifying to record that because of the increased energies of organizations, such as the PCI, this exciting twentieth-century building tool is developing refinement and stature as a construction method, so that eventually the architect will look to prestressing for his important projects with the same ease and confidence that go into selecting 2x4 studs for the small frame dwelling.

It is apparent that much money is being invested in research. For instance, the University of Illinois and the University of California all have major research projects on the books.

The multi-million dollar PCA Strength and Fire Testing Laboratories in Skokie, Illinois are providing data necessary for the ultimate and confident use of prestressed members and thin shell construction.

And the science of prestressing is becoming a mandatory part of the curriculum in most engineering schools. As a matter of fact, in establishing a new course in the science of building construction at Pratt Institute next fall, I intend to solicit the aid of this Institute to be sure that the field problems surrounding prestressed concrete fabrication and erection are adequately presented to the students.

It would appear, at least on the surface, and judging from some of the most recent accomplishments in your field, that you have developed a structural technique that could place architecture on the threshold of a golden era of spatial concept. And I visualize that our joint efforts of the future will probably rival the grandeur and the scale of the ancient monuments, in which were utilized, to the fullest extent, the engineering techniques available at that time.

Which gets us back to our architect friends: I have just recited a number of buildings that have utilized prestressed concrete. And I have touched upon the research and the educational activity being conducted by the industry, and yet the intimation is clear that wholesale acceptance of prestressed concrete by the architectural profession still needs some well-directed spadework.

As mentioned earlier, there are specific conditions that must be satisfied, questions to be answered, practical considerations to be over-

come.

In this talk, I would like to do three things. First, I would like to review some of the specific questions about prestressing that occur to the average architect.

Secondly, I would like to show by means of a few slides why we think prestressing is here to stay.

And, finally, I would like to suggest a program of education and public relations directed to the architect and designed to promote the increased use of prestressed concrete in architecture.

For the first consideration, I think the following are typical of the questions that are likely to be asked.

Will prestressed structural systems satisfy client need for flexibility? If reference is made to unencumbered floor areas, the answer, of course, is "yes." There can be no doubt as to the advantages of column free interior space, which provides complete flexibility in the planning or the re-planning of functional arrangements, whether they be office layouts or manufacturing facilities.

This becomes one of the most telling arguments in favor of prestressing and an advantage which any client is happy to adopt. However, flexibility suggests also physical change, such as expansion of facilities, either vertically or laterally, and very often this becomes important in the selection of the structural method.

I do know that extensive work has been done in the development of structural connections, and it is probable that expansion can easily be provided for. However, if there are standards, it would make that task much easier.

And another question: Can prestressed concrete be adapted to other building essentials, such as mechanical equipment, air duct runs, piping, electrical lighting and power distribution? As a matter of common sense, I would say that this type of accommodation is entirely possible, and even practical. We touched on this subject in designing the prestressed girders for Seattle's Norton Building, in which openings were cast—not over the columns, by the way—to allow the passage of various items of mechanical and electrical equipment, much the same as in steel framing.

But, going a step further, in considering the use of post-tensioned box girders for another project, we wondered whether these box girders could not double in brass as air ducts. The idea was abandoned because of difficulties at the connection. But I'm sure that some way will be found to accomplish this type of thing.

Could, for example, hollow prestressed floor slabs be adapted to radiant heating? Or could hollow prestressed exterior wall members, besides serving as girders, provide heating and also contribute insulation benefits? Perhaps your industry has a real bonanza in the making if the mechanical and the electrical engineers get into the act and aid in furthering your technical approach to the science of building construction.

Or an architect might ask: Will prestressed concrete satisfy the building codes to the extent that it doesn't precipitate major campaigns to allow for its use? I understand that steady progress has been made in this direction, and that your organization has waged an aggressive and continuous war against obsolescence in building codes where they affect prestressed construction.

You had a tough fight in Chicago where, thanks to the unceasing ef-

forts of your people—and, incidentally, a handful of progressive architects—the building department capitulated to the extent, as announced just thirty-three days ago in the *Chicago Daily Tribune,* of allowing prestressing in that city for most types of construction. The City Building Commissioner's acceptance was qualified, however, to exclude, and I quote: "Industrial plants where there is more than the average risk of fire."

But on a broader scale, your work on a uniform building code is producing dividends, particularly in the far west areas of this country.

This missionary work is most important to your industry, especially when you consider statistics covering building construction in, let's say, the New York area. In 1959, over two billion dollar's worth of construction was committed in this area. And assuming that 20 per cent of this volume involved structural work, it is obvious that four hundred million dollar's worth of business was enjoyed by the conventional structural interests, but because of our local code restrictions against prestressing, your industry didn't share a nickel's worth of this activity.

And what's just as bad, some 2,000 registered architects in this area have been deprived of the opportunity to experience the use of this marvelous structural technique.

Furthermore, although prestressing is technically allowed in many major cities, there are several important urban centers whose building departments do not go out on the limb of blanket approval. These bodies issue permits based upon the shopworn phrase of "individual merits of the case."

Eventually, as your enlightening work proceeds, building permits—excluding exceptional cases, of course—will not be based upon individual merits, but upon recognition of the sound engineering offered by the prestressed concrete industry. The sooner this becomes an accomplished fact, the better. For although the professional engineer and the architect can understand and cope with obsolescent codes, the layman client who is spending the dollars sometimes has disquieting feelings if his project doesn't know the full blessing of the local building department.

Another logical question: Will prestressing satisfy fire prevention criteria, and keep the insurance companies happy? A quick answer will produce a great big "yes" to this and, yet, Chicago's recent capitulation refers to fire hazard. And this along with the bonding of reinforcement issue is behind the New York City Code resistance.

The fire resistance testing being conducted in Skokie and other places should soon provide the answers to these problems, the successful conclusion of which should go a long way toward establishing some sort of uniform reaction as to insurance rates and classifications. As it now stands, the insurance companies are lacking much of the information they require with which to establish ratings and, as in the case of the several building departments, judge each case on its individual merits.

The architect asks: Does the use of prestressed members in system save construction time? It might be argued that shop fabrication is a time saver in any industry, and that in the case of prestressing shop methods can be and often are transferred to the field, yielding an on-the-site dividend, particularly where there are shipping and delivery problems.

While other industries can boast a full shop fabrication, it seems to me that you have a real plus when prestressed units are designed to serve as building components—such as walls, floor and ceilings—so that once a unit is put into place, it will serve also as a finished surface. Subject to correlation of mechanical and electrical appurtenances, this can effect real savings in construction time.

Question—and this is another $64 question: Is prestressed concrete economical for building construction? It wouldn't be fair to say "yes" or "no" until the conditions have been qualified and analyzed. While it may be more expensive per square foot to produce a building with 70-foot spans and prestressed than to produce the same building in 20-foot spans using structural steel, the values accruing from the long span solution must be reckoned in terms of the total worth of the building to the client. Actually, the experience record is still too brief to draw conclusive evidence, since geography and other influences should be taken into consideration.

As an example, the Hawaiian shopping center referred to before would not be feasible in structural steel due to the high cost of shipping steel overseas and so steel, obviously, was not in the picture; whereas, cost analysis is in order before one could select prestressed concrete in favor of structural steel in, say, Harrisburg, Pennsylvania.

It is evident, however, that part of the important effort being expended by your industry is devoted to finding ways and means to make prestressing palatable dollar-wise to building owners.

The architect might ask: Does prestressing yield dividends in so far as precision work and quality of workmanship are concerned? A strong affirmative is to be found in the adoption of factory methods, whether members are pretensioned and post-tensioned in the shop or in the field, since the basic methods of prestressing do not vary with location. It is evident that all architects are deeply interested in the product that yields finely controlled surfaces on which develops the ultimate in efficiency and function from the compressive virtues of high-strength concrete.

In fact, the handsome maintenance-free surfaces of prestressed concrete appeal to the practical instincts of the architect and his client.

And now, a long-hair question: Does prestressing satisfy the aesthetic in architecture? This is one question I feel can be answered as well by the architects as well as by the proponents seated in this auditorium.

It is my opinion that prestressing may have more architectural significance than you suspect. Because of this medium, the architects will have a tool that will enable them to produce structures rivalling the stature and breathtaking scale of the ancient great buildings. This will require bold, broad, articulate planning and close cooperation between engineers, manufacturers, contractors and architects, coupled with an intense desire to be major league in the use of this technique.

I sincerely do not expect that our profession will settle for only minimal uses of prestressed concrete, the creation of only nominal enclosures, and the indulgence of only uninspired solutions. I fully expect our enlightened architects to seize upon this great facet of technology and produce significant architectural and engineering art forms.

This approach has plenty of his-

torical precedent. The Greeks had only stone columns and lintels to work with, but they made the system noble in concept. The Byzantines and the Romans and the Gothic peoples learned how to put little stones together in arches and span space that would rival much of our present-day steel and reinforced concrete work.

When you walk into St. Peter's in Rome, your reaction has to be one of admiration for the builders who wouldn't settle for the mini-

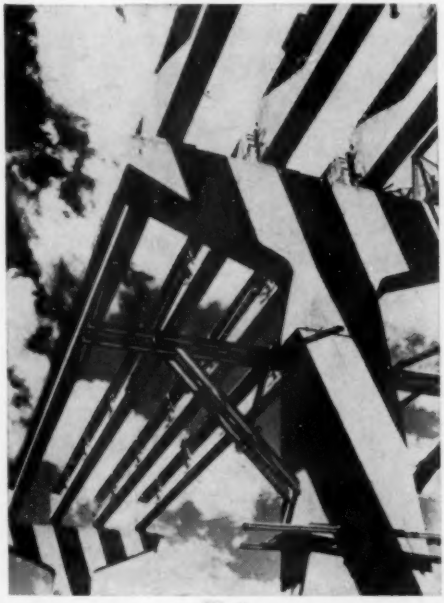


Fig. 1

mum advantages of the engineering tools placed at their disposal.

And thus in prestressed concrete, I think we, working together, have a potential threat to mediocrity in structural expression. Yes, the prestressed concept can and will produce results of great aesthetic appeal.

And, after all, part of our job as architects is to produce buildings that look good as well as being functional. If we can combine function with beauty and arrive at that happy state through the use of a material that will serve both, we are nearing the ultimate in architectural thinking.

Fig. 2

We know that your product will lend itself to this end and, as your consumers, it is up to the architect, with your helpful guidance, to develop further the examples of imaginative prestressed architectural accomplishment, already a matter of record in France, Denmark and Russia. For, on first inspection, it would appear that as usual Europe with its cheap labor is miles ahead of this country in the use of prestressed concrete. This was undoubtedly true in the earlier days when its use in this country was not

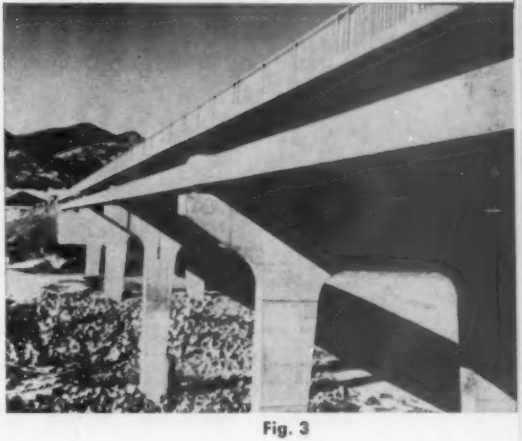


Fig. 3

encouraged by a construction industry that thrived on cheap steel and inertia.

The Korean War, with its resultant shortages of steel and an all-out superhighway program, provided the springboard from which your industry has really taken off. So with your house in order, boasting sound financial status, committees working in many directions to improve the many aspects of prestressing, you are now ready to invade in a big way the building construction field.

With the cooperation of the prestressed concrete industry, I promise you there is enough architectural talent in this country to produce a wave of prestressed architecture second to none in the world.

I should like now, if I may, to show you some illustrations of the use of prestressing as it contributes to architectural expression and function. And I would ask you to observe these in the spirit of whether or not prestressing reads as the basic structural element, rather than deciding whether they suit your own particular aesthetic taste.

With the exception of the high school auditorium, which you will shortly see, these are some of the modest efforts which the office that I am associated with has produced.

Figures 1, 2, and 3 show one of the bridges designed in connection with the Air Force Academy. The design of simple span bridges such as this can no longer be considered to be extraordinarily newsworthy, in so far as the prestressed interests are concerned. This in a way provided us as architects with a sort of basic training in the use of the material and, as you can see, the results are attractive and monumental in character. In addition, these bridges provided us with primary knowledge of the problems inherent in the field handling of enormous structural shapes.

Figures 4, 5, 6 and 7, illustrate the plans and elevations of an administration building which is currently under construction for United Airlines, near O'Hare Field, Chicago. This building, incidentally, was involved in the discussions between the Chicago Building Department and the prestressed concrete interests. You will note from the plan that the 66-foot by 60-foot bays made possible by prestressing produced nearly column-free interiors, and are a function of the building module that best suits United's operations, whether the demand be for open pool space, private offices or meeting rooms. Incidentally, the area to the right—which is shown now as a cafeteria and kitchen complex in the floors above—is completely wide open and is used by United as open pool area.

The detailed wall section shown in figure 7 illustrates how the exterior wall has been set back from the exterior columns to reduce glare and provide permanent window washing and maintenance platforms. It illustrates, also, how completely free of the skin enclosures and of the other appurtenances the prestressed frame is. You will see in subsequent photographs that the whole structural aspect of the building is expressed and that the frame itself has been subjugated to the structural elements of the building.

Figures 5 and 6 are pictures of a model of the building, and the elevations are characteristic of the long low span construction offered by prestressing, and show immediately that it is possible to have unencumbered floor space without the cost penalty of excessive depths that conventional structural meth-

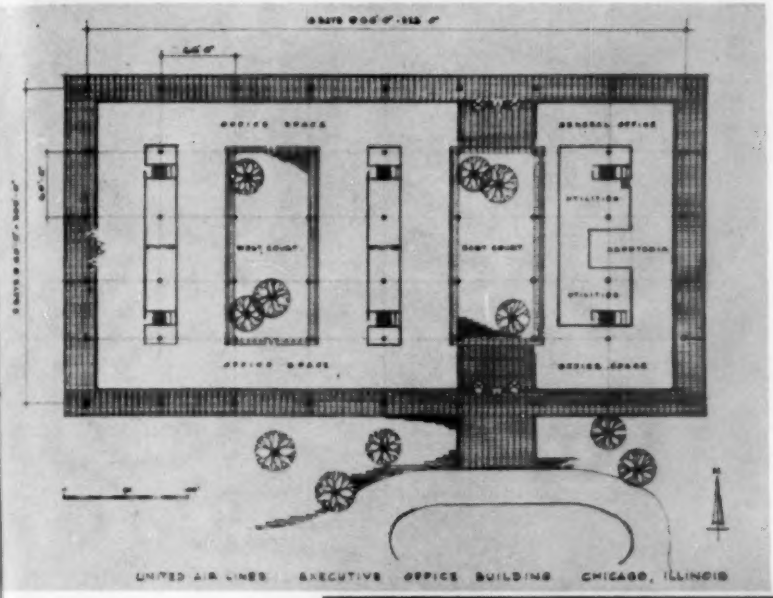


Fig. 4

Fig. 5

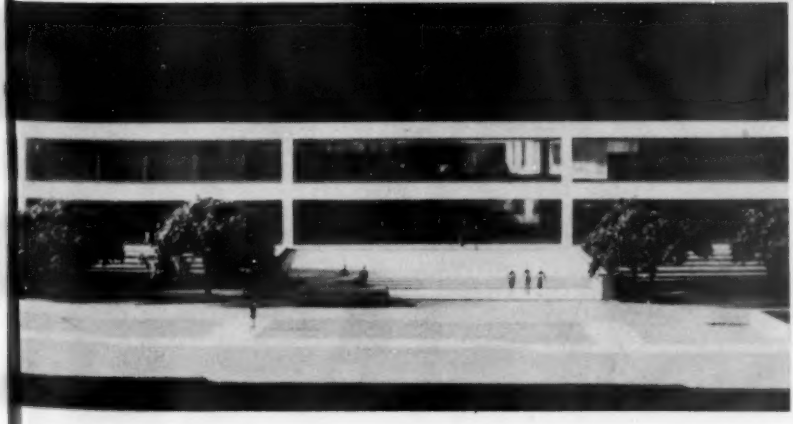


Fig. 6

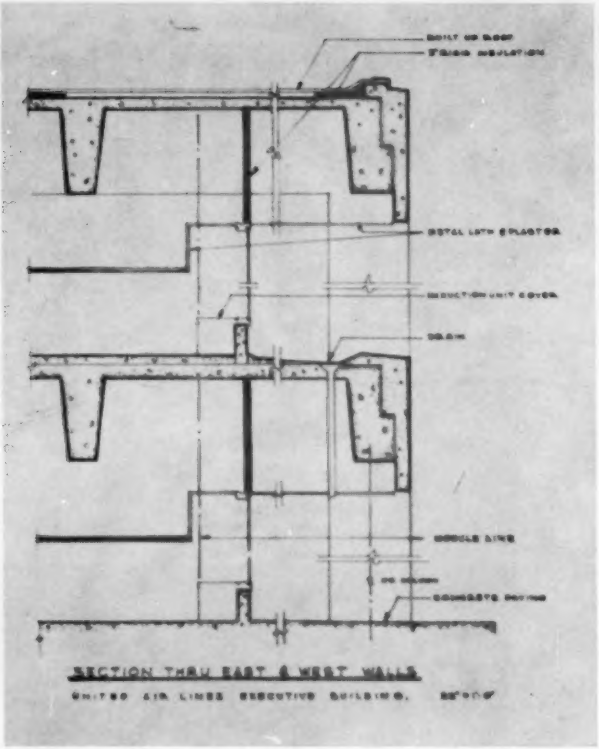


Fig. 7

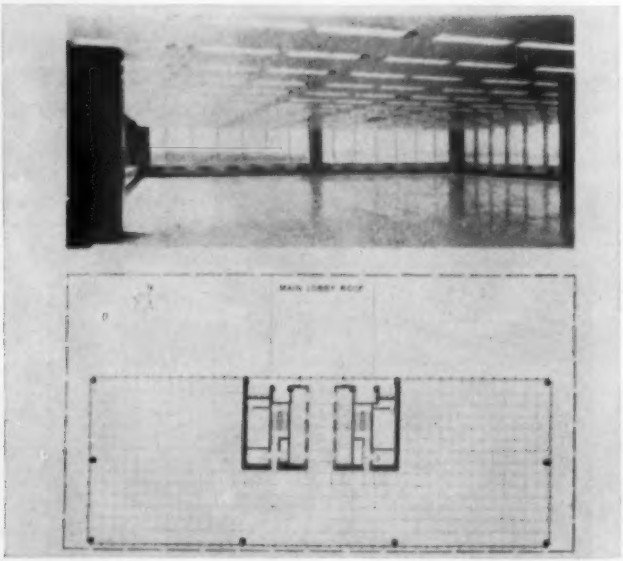


Fig. 8

ods would require to offset bending and deflection. The framing members were all post-tensioned at the site. The architectural result is a long low, extremely graceful building.

Figure 8 is a plan and photograph of a typical floor in the 21-story Norton Building in Seattle, Washington. We were given a program by the owner which required as near 100 per cent rental area as possible. This was made economically feasible by combining 70-foot prestressed concrete beams with steel column girder frame, so that the net result is a column-less interior.

Figure 9; a photograph of a prestressed girder being hoisted into place illustrates the method used for adapting prestressed beams for mechanical and electrical utility runs by creating openings in the webs of the beams.

Figure 10 is a study in prestressed concrete for a convention hall that was to have been constructed on the West Coast. It never got beyond the drawing board stage, the West Coast interests having other ideas as to choice of materials. Taut as a drum, this thin shell roof and arena structure, 400 feet in diameter, shows the slimness, grace and long spans possible in prestressed concrete.

Figure 11 is a photograph of a 146-foot prestressed girder being placed during the construction of a high school auditorium in Springfield, Illinois. This illustrates one ingenious solution to the problem of erecting very large and heavy members, especially when there is some height involved. As you can see, the architect-engineer-contractor team combine their efforts to produce a structure that would lend itself as a hoisting tower. The power is supplied by the prestressing jacks.

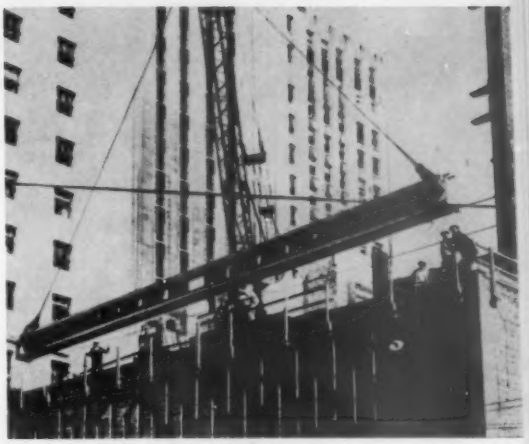


Fig. 9

As a result (fig. 12)—and this is the same auditorium—a handsome column design has emerged from the functional necessity and fits in aesthetically with the equally handsome prestressed girders.

The last project I will present represents exploration into rugged, honest, almost monumental expressionism, utilizing to the utmost the virtues of prestressed concrete. The floor plan (fig. 13), as you see, is disarmingly simple and features 70-

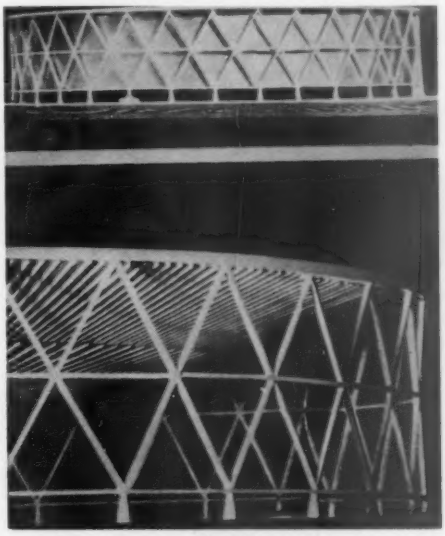


Fig. 10

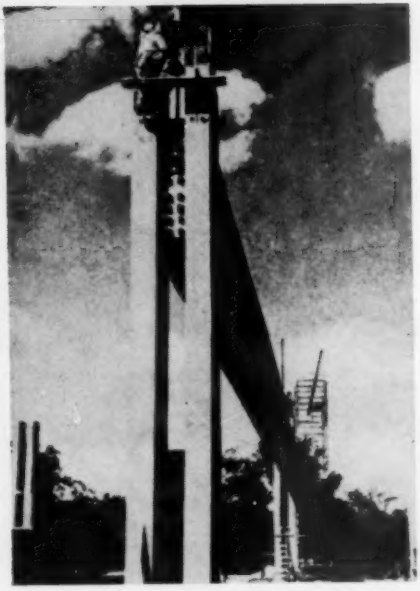


Fig. 11

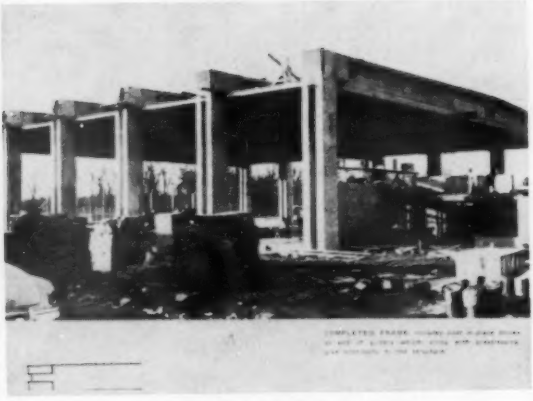


Fig. 12

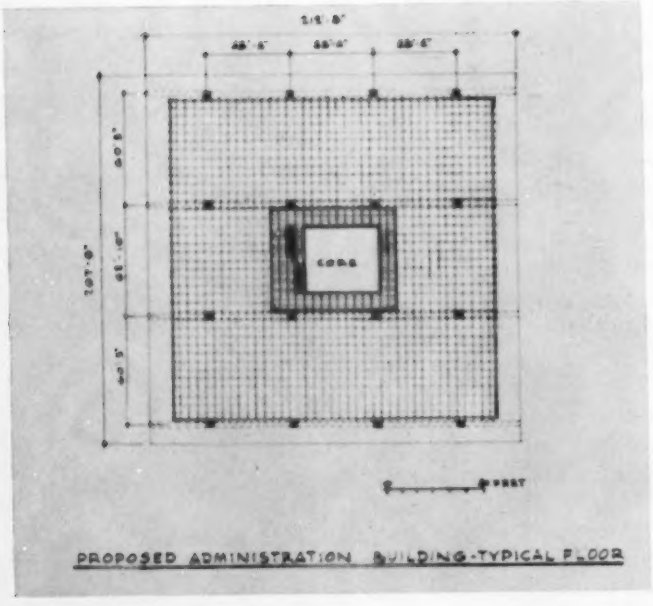


Fig. 13

foot spans and 30-foot cantilevers, all of which would be deflection free. The beauty of the large open floor areas and the column-free exterior curtain walls is matched only by the exterior appearance, which can only be a product of prestressed design.

The two elevations shown in figure 14 indicate the varying characteristics of the structural system as expressed on the exterior. Because of the overhangs and cantilevers, the resultant play of light and shadow suggest some of the monumentalism of the ancients. This is suggested in the enlargement of the elevations in fig. 15. It should be emphasized that—this is actually the same building—each figure containing one side and one end elevation. Basically, this is a structure of prestressed box girders running longitudinally—70-foot on centers—and are surmounted by T-shaped prestressed beams which are 100 feet long, and which span the building and then cantilever beyond the end box girder. The beams being T-shaped, provide a surface for the floors and ceilings.

In one view, we see the T-shaped beams in elevation and we see the ends of the girders, or the section of the girders, and we get one type of structural expression—very rugged and very simple.

In the other elevation, we see expressed the projection of all the T-shaped beams and their part in the light and shadow effect of the structure.

The main supporting members, as I said, are 5-foot square post-tensioned box girders upon which are supporting the T-shaped beams, 100 feet long, and they are shown in the isometric diagram in fig. 16.

The prospective sketch in fig. 17 shows how powerfully and beautifully the structure can be expressed. This is post and lintel architecture in its most modern form. Every one of the elements that you see there that have any strength in any dimension, in any size, and which contribute to the chiaroscuro of a building are structural. There has been no attempt, nor would there be any attempt to try to conceal any of this structure.

The examples you have just seen represent, we feel, a very modest prelude to the exciting architectural achievements to be developed using prestressing. None of these can be classified as flamboyant in the sense that the well-publicized Congress Hall of West Berlin is famous, but they do set forth intriguing solutions for the typical everyday problems which comprise 98 per cent of the average architectural problems in this country, and which, in turn, represent a comparable percentage of the overall dollar volume.

We are, therefore, anxious to continue to produce inspired buildings within the limits, however, of functional and economic precepts that always must accompany any program leading to fine architecture.

Earlier, I had used such words as "education" and "public relations." I am very serious in suggesting to you that you establish a comprehensive program directed towards acquainting architects, designers and potential clients with the wonders of prestressed concrete, for I do feel that most of us have not been privileged to fully understand its potential.

This is understandable, since your growth has been so fast and so recent that first things have had to come first. But I remember in the thirties, when glass block first reared its ugly head. The architects, engineers, builders, owners, and profes-

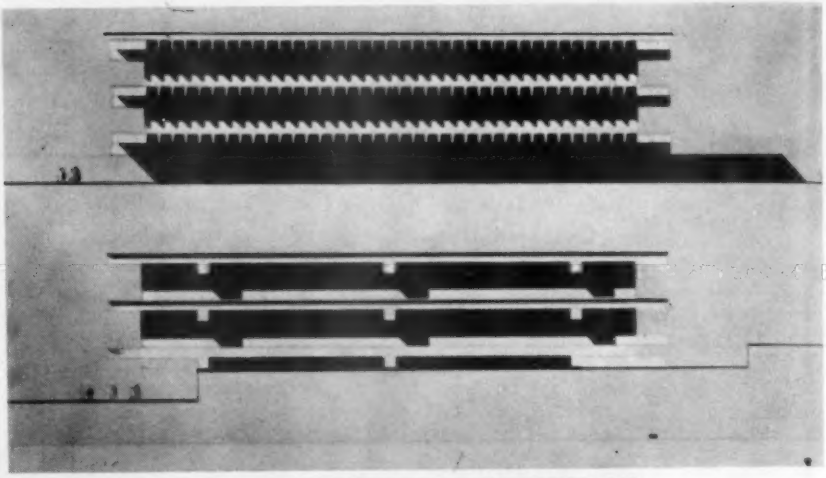


Fig. 14

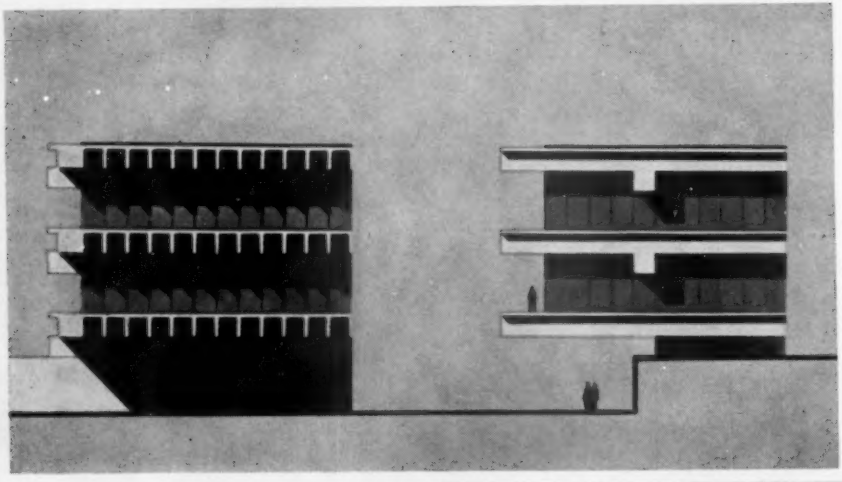


Fig. 15

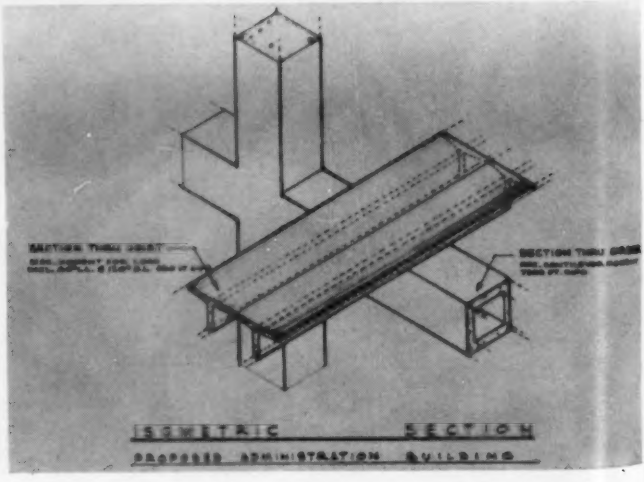


Fig. 16

sional schools were flooded with litterature, pictures and samples by the glass block interests. Architectural competitions were held for the design of homes, office buildings, institution buildings, and a dozen other categories. Distinguished juries made the judgments and prizes were awarded. The result: America was deluged with glass block architecture. Of course, the fact that this material is practically extinct today is quite beside the point. I have reason to believe that prestressing will always be with us.

What I am suggesting is that you create a facility for bringing prestressing to the architect. I'm not in a position to tell you what that organization or form of this information dispensing agency ought to be, but it shouldn't be too difficult to decide as an industry in what direction your interests may lie and act accordingly. Maybe a few regional information centers might be in order where the dozens of questions, of which I have presented a few, might be answered. Or perhaps a committee governing news releases and non-technical brochure type information might justify its existence.

Perhaps it would be well to think seriously of architectural competitions. Arrange inspection and field trips for architects. These activities could be controlled by a select committee on which at least one architect could serve.

It would be mutually advantageous to the prestressed industry and to the architects if no stone were left unturned in getting the message across to the architectural profession—from the students to the seasoned practitioners. For I am sure that your public relations people will agree that wide professional acceptance, based on knowledge of a technology such as yours, will pave the way on your behalf to easier relationships with building departments, insurance underwriters, and, God bless them, our long-suffering clients.

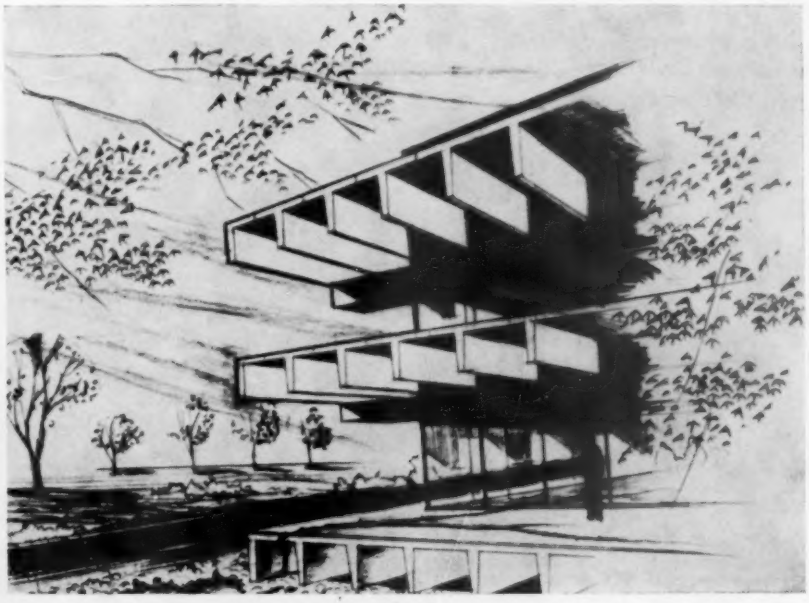


Fig. 17

Behavior of Prestressed Concrete Under Dynamic Loading

By

P. W. Marshall[1] and A. M. Ozell[2]

Very limited information is available on the behavior of prestressed members under dynamic loading. Although many prestressed structures or their component parts presently in use, such as power poles, railroad ties, machinery supports and the like, are subjected to dynamic and vibratory loading.

Tests reported herein were undertaken to study the various characteristics of prestressed concrete under such conditions. The specimen used in this study was a diving board, designed and constructed by the civil engineering students at the University of Florida, which was tested under dynamic and static loads. To the authors' knowledge, such a study is believed to be the first of its kind ever reported in the literature.

The board was 20 in. wide, 2-5/8 in. thick, tapering to 1 1/2 in., and 16 ft. long. The free end had an 11-ft. cantilever. It was pretensioned with six 7/16-in. strands spaced at 3 in. on centers and positioned 1 in. from the top surface. Transverse reinforcing was accomplished by coat hanger wire spaced at 8 in. throughout. The concrete mix was designed for a 28-day strength of 8,000 psi and had 7,140 psi cylinder strength at release at four days (8,700 psi in 28 days). Concrete was internally vibrated and wet burlap cured until the stress release of the strands.

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(2) Professor of Civil Engineering, University of Florida, Gainesville, Florida.

Figure 1 shows the construction layout of the diving board and Figure 2 the completed structure ready for testing.

Dynamic Modulus from Natural Frequency

The following discussion concerns the determination of the dynamic modulus, E, for the diving board in question. The natural frequency was determined manually by counting the number of cycles occurring within a timed interval. Several determinations were made, yielding an average frequency of 3.6 cycles/sec. A mathematical solution of the diving board as a vibrating beam is necessary for the interpretation of these data.

The basic differential equation for a vibrating beam is:

$$EI\frac{d^4y}{dx^4} = uyw^2$$

where E = Young's modulus
I = moment of inertia
y = deflection at any point x on the beam
u = mass per unit length
w = angular frequency of vibration = $2\pi f$, where f is frequency of complete cycle.

The general solution to this equation is:

$$y = Ae^{ax} + Be^{-ax} + C\cos ax + D\sin ax$$

where $a^4 = \frac{uw^2}{EI}$. However, since this form is quite cumbersome for other than the simplest uniform beams, a numerical procedure was used for the calculations in this discussion.

One numerical method of analysis

Fig. 1

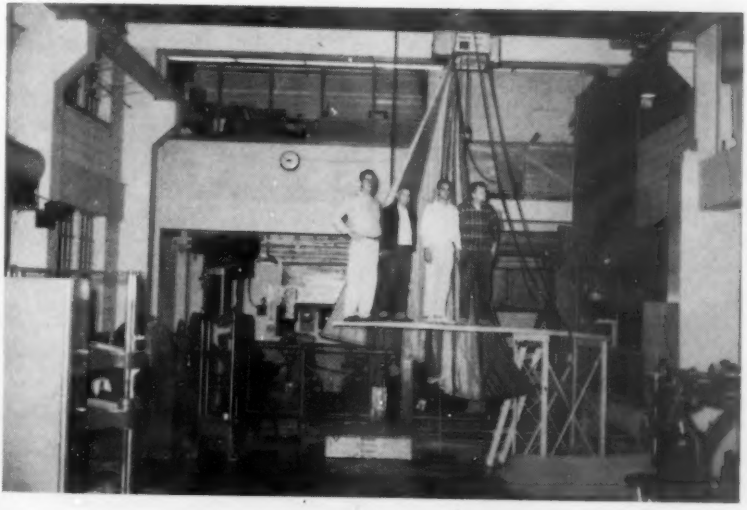


Fig. 2

for the deflections is Stodola's iteration procedure in which the total mass is divided into a finite number of concentrated masses with short sections of weightless beams connecting them. Maximum correspondence between the actual and equivalent beams will occur when the number of segments is large but the difficulty of finding the solution increases.

In this study 1-ft. segments of the diving board with measured values of section properties were used. A dynamic load distribution was estimated and the problem was solved statically for the resulting deflection curve. This should be closer to the actual first mode vibration than the first assumption and with a new dynamic loading.

$$W = uy\,(w)^2,$$

a better approximation of the actual deflection can be obtained. This process can be repeated until the desired degree of accuracy is obtained. For this study, three cycles of calculation were performed and a deflection curve related to unit deflection at the toe of the diving board was obtained.

Now the total kinetic energy,

$$K_k = \tfrac{1}{2}\int_0^L u\,(2\pi f y)^2\,dx,$$

can be evaluated numerically from the calculated deflection curve and known section properties in terms of frequency f. Also, total potential energy,

$$K_p = \tfrac{1}{2}\int_0^L EI\left(\frac{d^2y}{dx^2}\right)^2 dx,$$

can be evaluated from the deflection and section properties, but in terms of E. Since all parts of the beam undergo simple harmonic motion, it is possible to equate the two kinds of energy and obtain an expression relating elastic modulus to frequency.

From this expression and the measured frequency of the prestressed concrete diving board a value for the dynamic elastic modulus of 6.2×10^6 psi was calculated. Some of the calculations are shown in Figure 3.

High Frequency Response

In order to obtain more specific data about the elastic properties of the diving board its amplitude response to various driving frequencies was obtained. Although studies with the primary mode of vibration would have been desirable from the standpoint of direct application to springboard characteristics, the limitations of available means forced the studies to be conducted with the second mode. The Lazan oscillator which was used would not run slowly enough to excite the first mode and manual driving tended to follow only the resonance frequency.

The results of the high frequency studies are presented, in part, in Figure 4. Frequency was read from a tachometer attached to the oscillator but was relatively inaccurate. A stationary node formed about 2 ft. from the toe of the diving board. Amplitudes measured at the toe and at 7 ft. from the toe showed very similar patterns for the second mode.

The asymmetrical frequency response near the second mode suggests a "soft" nonlinear stress-strain relationship. The apparent discontinuity at the second mode indicates that the damping is quite light.

Summary of Elastic Properties

Values for the elastic modulus of the concrete used in the diving board were found to be:

Distance from Toe, ft	Unit Static Deflection	Unit Vibrational Deflection	$\frac{\Delta^2 y \times 10^6}{\Delta x^2}$ (Vibrational)	"Measured" I, in.^{4}
0	1.000	1.000	0	7.3
1	0.876	0.885	5	9.8
2	0.756	0.773	13	14.1
3	0.639	0.662	20	19.6
4	0.527	0.553	30	23.7
5	0.422	0.448	41	26.2
6	0.326	0.350	49	30.8
7	0.238	0.258	56	35.6
8	0.160	0.175	73	34.5
9	0.092	0.102	98	31.2
10	0.039	0.043	112	32.3
11	0.000	0.000	127	33.0
12	−0.022	−0.024	117	28.8
13	−0.029	−0.032	81	31.2
14	−0.025	−0.028	54	31.2
15	−0.014	−0.016	28	29.8
16	0.000	0.000	0	30.8

Kinetic energy: $K_k = \frac{1}{2} \int_0^L u\,(2\pi f y)^2\,\Delta x$

$$g \int_0^{16} u\,y^2 \Delta x = 132.2 \text{ in.}^2\text{–lb.}$$

$$f = 3.6 \text{ /sec}$$

$$g = 384 \text{ in./sec}$$

$$K_k = 87.5 \text{ in.–lb.}$$

Potential energy: $K_p = \frac{1}{2} \int_0^L EI \left(\frac{\Delta^2 y}{\Delta x^2} \right)^2 \Delta x$

$$\int_0^{16} I \left(\frac{\Delta^2 y}{\Delta x^2} \right) \Delta x = 28.25 \times 10^{-6} \text{ in.}^2$$

$$K_p = 14.12 \times 10^{-6}\ E \text{ in.–lb.}$$

$$E \text{ (dynamic} = \frac{87.5}{14.12 \times 10^{-6}} = 6.2 \times 10^6 \text{ psi}$$

Summary of Calculations for Dynamic Modulus

Fig. 3

test cylinder $= 4.4 \times 10^6$ psi
static load $= 6.0 \times 10^6$ psi
vibration $= 6.2 \times 10^6$ psi.

Since each of these values was obtained from a single determination, the reliability of any one value taken alone is questionable. However, since the two values taken from the diving board itself were in general agreement, they may be used, with caution, in further studies of the dynamic behavior of this prestressed concrete member.

Generally, the elastic properties of the board were nonlinear for both static and dynamic tests. For small-amplitude high-frequency vibrations poor damping was noticed.

In the dynamic studies to follow, stresses were calculated on the basis of observed deflections. For a 1-in. deflection the following stresses are induced under the stated conditions:

Initial Tangent to static load deflection = 22.8 Kip.-in.
first mode vibration (calculated) = 25.2 Kip.-in.

The difference between these values was caused by different shapes of the deflection curve. Approximately a 10 per cent error will result from the use of the static load-deflection curve for stress determinations. However, this method has the advantage that the data is available, and it is felt to be more reliable than the linear extrapolation of small deflection moduli to the large deflections encountered.

Dynamic Behavior in Use

It was noticed that the diving board's action was tricky. A detailed study was made of its motion under operating conditions and several surprising results were obtained.

A diver was asked to spring vigorously from the board to excite the oscillations that would occur in use. The divers motion and that of the board were recorded by a 16-mm camera operating at 64 frames/sec. Because of the divers familiarity with the board he was able to maintain the large steady-state ampli-

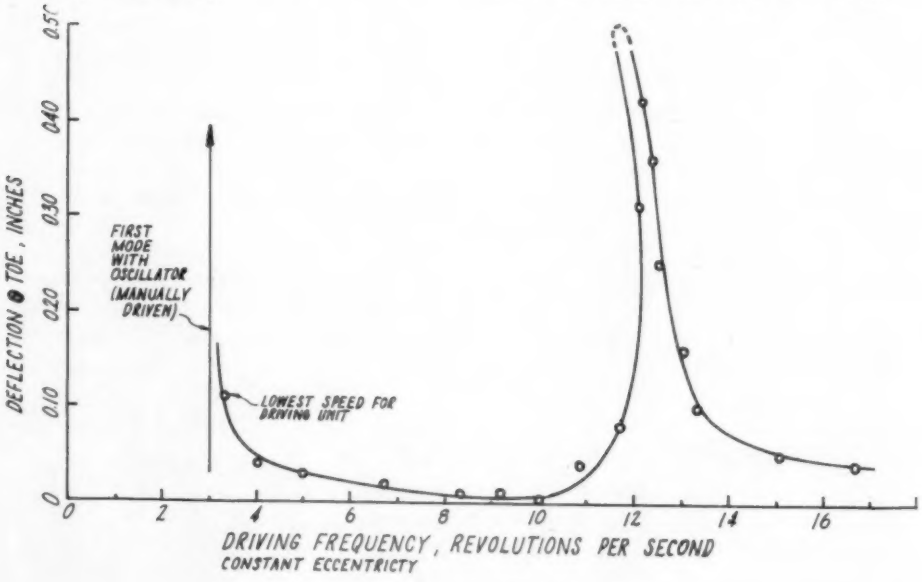


Fig. 4

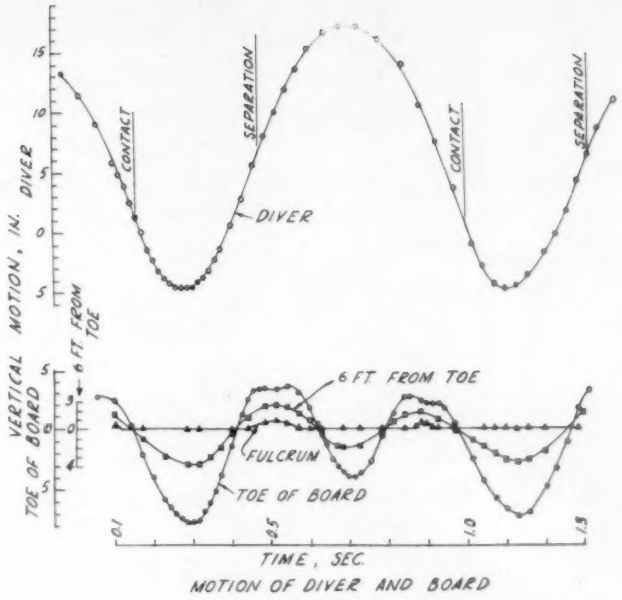


Fig. 5

tudes studied.

A time-and motion study projector was used in the analysis of the films. The pictures were analyzed individually as projected on a rectangular grid. Vertical positions could be read to 0.1 in. Time control came initially from assuming that the camera speed was accurately 64 frames/sec; however, experience indicates that the camera speed varied.

The data for a typical jumping cycle are shown in Figure 5. Time was plotted initially as frame number, and vertical position as grid ordinant. The velocity and acceleration of the diver at various instants were obtained by successive numerical differentiation of the plotted deflections. Conversion of the results to conventional units gave the average acceleration of the diver in free fall as 28.2 ft/sec^2. This value becomes 32 ft/sec^2 if the true camera speed is taken as 69 frames/sec. The accelerations plotted in Figure 6 have been adjusted on this basis; gravitational acceleration is taken as the standard.

Discussion of the Results

The deflections of the board show several interesting actions. While the diver is acting on the board, the toe deflects 7.0 in. below the static deflection of 0.7 in.; the magnitude of this deflection is considerably more than anticipated. The downward deflection of the toe takes 0.26 sec., which time corresponds to a frequency of 1.9 cycles/sec. This slowing down is due to the combined effects of the divers mass and the nonlinearity of the stress-strain curve.

While the diver is in midair the toe of the board oscillates with a fundamental frequency of 3.7 cycles /sec., and with a secondary mode of four times this frequency. Comparison of the toe and the 6-ft.-point deflections shows that these two points are in phase for the fundamental mode but out of phase for the secondary mode. These two modes correspond to those studied

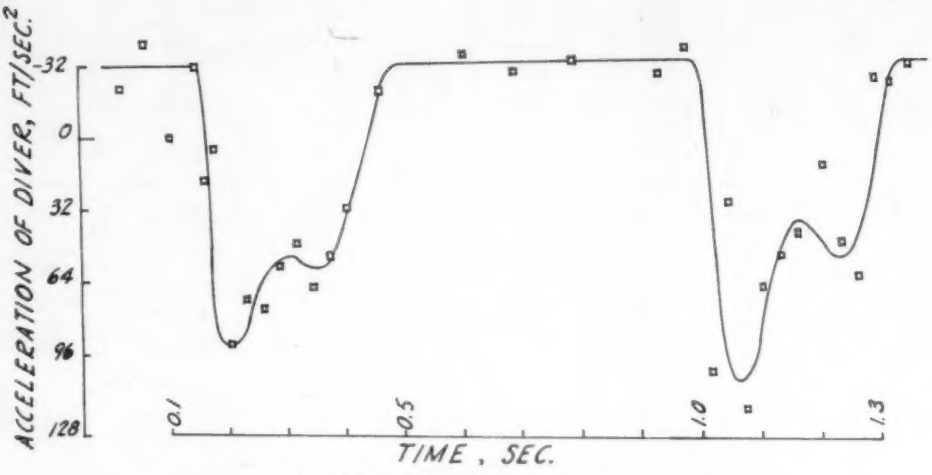


Fig. 6 - ACCELERATION OF THE DIVER

previously. The secondary mode could be definitely felt by a person standing on the board. The upward momentum of the board caused its fulcrum to leave its support for a short time, coinciding with a reinforcement peak of the two vibrational modes. This combined motion gave the appearance of a wave traveling the length of the board and caused excitation of the cross-bracing in the supporting frame.

Because of the large mass of the prestressed concrete diving board and its poor damping, it was necessary for the diver to coordinate his movements closely with those of the board. Notice that his cycle corresponds to two cycles of the board; he must manage to make contact with the board just as it starts downward. It the diver hits the board going the wrong way it acts as a massive club and jolts him to an abrupt stop. Coordination with the board while building up its amplitude is even more complex than maintaining a steady state.

While it is realized that to a well-coordinated person the complex motions described above soon become instinctive, they do present a problem to one not familiar with the unusual action of such a diving board.

Observed Stresses

For purposes of design it is important to know the nature of stresses induced in the structure under investigation. For the diving board value for an impact factor of three was found to be in error.

The total live load and impact deflection was found to be 7.7 in. On the load-deflection curve this corresponds to a moment at the fulcrum of 96.0 kip-ft. Such a moment would be produced by an 800-lb. load acting 1 ft from the toe of the board. Since the acceleration of the diver accounts for only 500 lb. of this it must be concluded that the inertial mass of the board itself accounts for a large part of the stresses. An impact factor of 6.4 expresses the combined effects of the diver springing on the board.

The distribution of flexural stresses for various loading conditions is shown in Figurre 7. Dead load moment was calculated from the actual beam. Impact moments were based on test results. Moments due to first mode vibration were based on mathematical analy-

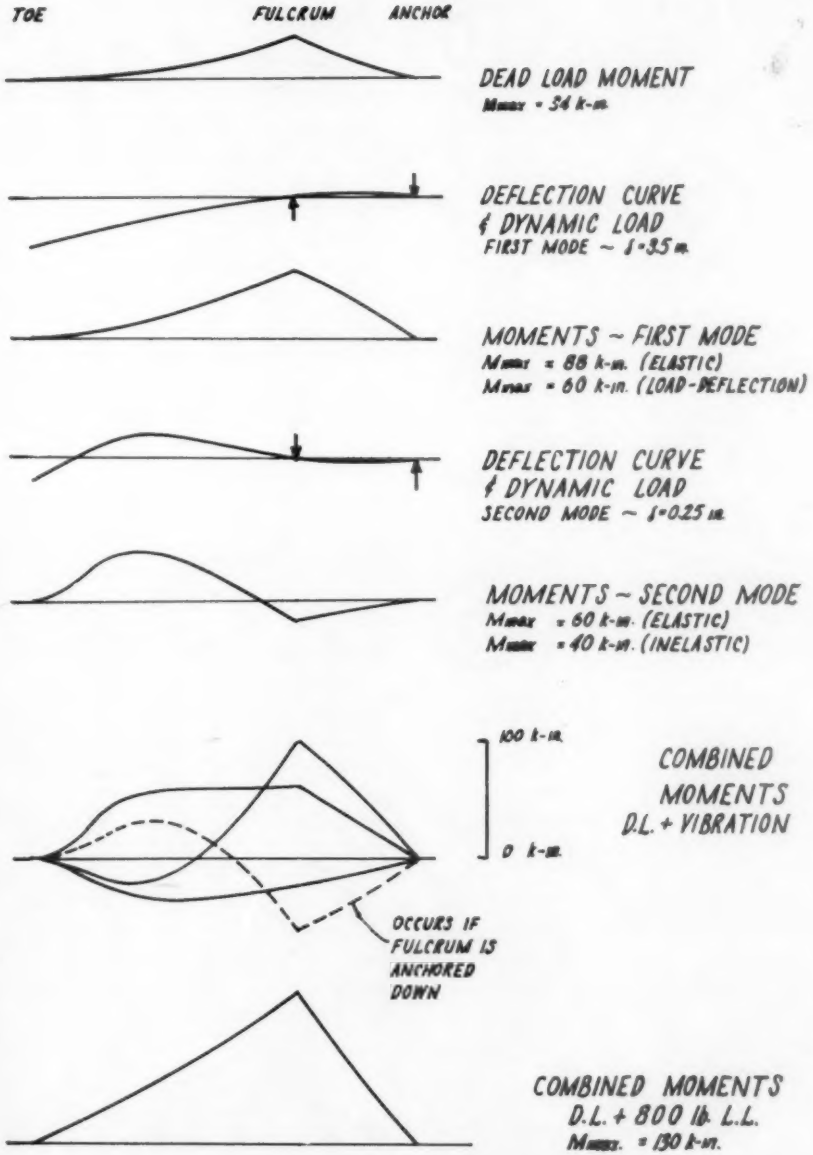
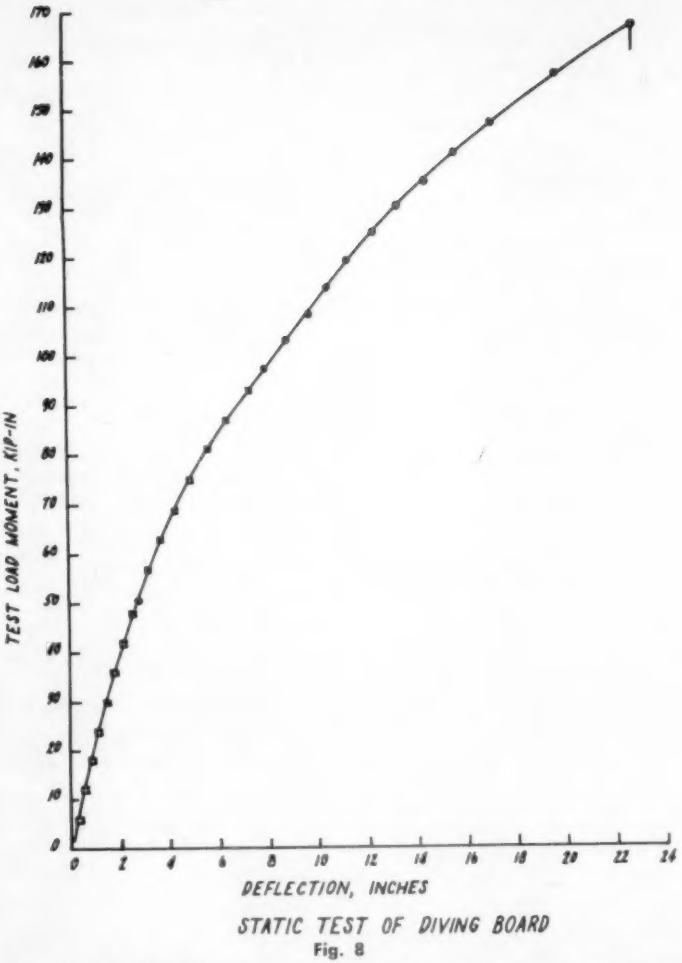

TOE
FULCRUM
ANCHOR
DEAD LOAD MOMENT
Mmax = 34 k-in.
DEFLECTION CURVE
& DYNAMIC LOAD
FIRST MODE ~ f=3.5 ~
MOMENTS ~ FIRST MODE
Mmax = 88 k-in. (ELASTIC)
Mmax = 60 k-in. (LOAD-DEFLECTION)
DEFLECTION CURVE
& DYNAMIC LOAD
SECOND MODE ~ f=0.25 ~
MOMENTS ~ SECOND MODE
Mmax = 60 k-in. (ELASTIC)
Mmax = 40 k-in. (INELASTIC)
100 k-in.
0 k-in.
COMBINED
MOMENTS
D.L. + VIBRATION
OCCURS IF
FULCRUM IS
ANCHORED
DOWN
COMBINED MOMENTS
D.L. + 800 lb. L.L.
Mmax. = 130 k-in.
~ FLEXURAL STRESSES (shown on tension side)

Fig. 7



STATIC TEST OF DIVING BOARD

Fig. 8

sis and the load-deflection curve. Moments due to the second mode were based on measured deflections and a rough approximate analysis. The combined moments are somewhat speculative and are based largely on qualitative observation of the diving board's behavior. Nevertheless, these diagrams give a better picture of stresses in the board than a single numerical value for maximum moment at the fulcrum.

Several features of the stress distribution deserve particular notice. The D. L. + L. L. condition was critical for flexure at the fulcrum, while combined D. L. plus vibration produced higher stresses near the end of the diving board. Stress reversal occurred to some degree everywere along the board but was most serious near midlength; an even larger reversed stress would have occured at the fulcrum had it been tied down.

Static Tests

The prestressed concrete diving board was subsequently tested to failure to check the actual ultimate strength with the calculated value. The loading was done by placing weights at the tip of the cantilever

span. Figure 8 shows the moment vs. deflection curve. The maximum deflection at failure was almost 23 in. at the tip of the 11-ft. cantilever end—equivalent to a concentrated load of 1,300 lb. placed at the end of the diving board. The actual ultimate moment of 167 kip-in at failure compared well with the calculated ultimate moment of 162 kip-in, using rectangular stress block, the latter based on a maximum compressive strain of 0.0030 in./in (obtained from cylinder tests), f'_c of 9000 psi for concrete and yield strength of 250,000 psi for the strands (corresponding to 0.011 in./in strain).

Figure 9 shows the compressive failure of the board at the fulcrum.

Conclusions

The dynamic modulus of prestressed concrete undergoing small deflections was found to be about 6.2 x 10^6 psi.

The impact factor was determined as being nearly 7. Design loads suggested for similar spans are 600 lb. for cracking and 2,000 lb. for ultimate load applied 1 ft. from the toe of the board.

From the observed behavior under dynamic and static testing of this diving board it is concluded that the mass of members subjected to similar loads has great effect upon their reactions to load.

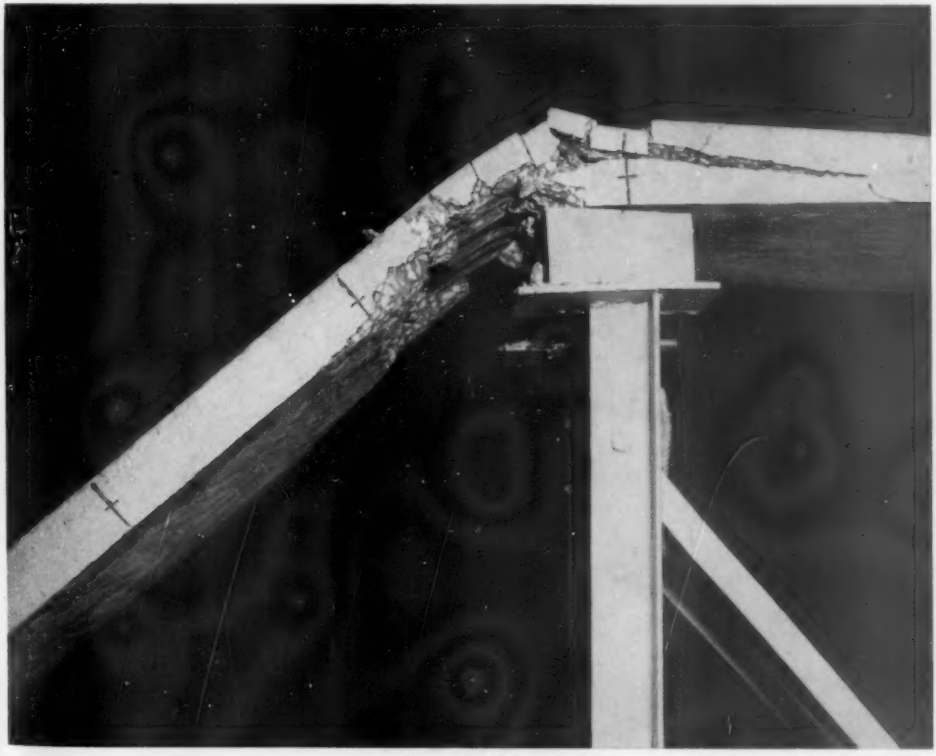


Fig. 9

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Europe's Contribution To Imaginative Engineering

by Daniel C. Vandepitte *

Presented at the Prestressed Concrete Institute Convention
New York City, September 28, 1960

INTRODUCTION.

It is the author's purpose to expound a few ideas which have found their way into European engineering practice and to describe various structures in which they are embodied. He certainly does not want to imply that these techniques are exclusively European, let alone that they are unheard of in the United States. Some may also have cropped up in American practice, perhaps even before they did in Europe. Neither does the author suggest that these ideas can all be used economically in the United States with its different materials and labor costs.

Most of the structures described as examples were built in Belgium. This fact is incidental and is not at all to be construed, of course, as a hint that Belgium should be regarded as the main source of creative engineering ideas. It is due simply to the author's greater familiarity with what Belgian engineers have accomplished.

The author is under the impression that prestressed concrete in the United States overwhelmingly means precast, prestressed concrete. This is by no means so in Europe and it will be noted that the techniques which are to be discussed relate mainly to post-tensioned concrete. The author also wants to state his belief that prestressing techniques are gradually growing more sedate and that they do not move forward any more with great leaps and bounds, as they did in the first feverish years of their existence.

Statically Indeterminate Structures.

One phenomenon worth noting is the constantly increasing importance of statically indeterminate structures. At the beginning of the prestressing era, almost all structures built were, quite understandably, statically determinate. But continuity is now a feature of a great many prestressed structures. Although the cost differential between a continuous prestressed structure and its reinforced concrete counterpart, is, in general, decidedly smaller than the one between a statically determinate prestressed structure and its reinforced concrete counterpart, supposing, of course, that such alternatives are feasible.

External Cables.

In Belgium, and also in Great Britain and Germany, many post-tensioned structures are prestressed by means of external cables or, more accurately, by means of cables that are not placed inside, but outside the web of I beams, or between the webs of box girders.

External cables are polygonal. Changes of slope are achieved around one end of vertical stiffeners that are cast with the web or

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around dowels that stick through the web. After tensioning, the tendons are embedded in mortar to protect them against corrosion. The change of slope of the cables at any one point usually does not exceed the changes of direction that occur in most anchorage systems. Since these are known not to decrease measurably the tensile strength of the wires, no such decrease occurs in polygonal external cables.

Inspection of many external cables has shown that the mortar surrounding them does not crack in spite of its shrinkage. This is due to the shrinkage and the creep of the prestressed concrete and to the high percentage of finely spread reinforcement in the mortar. In Belgium, a mortar cover of 1.4 in. is normally placed on the wires in structures that must be fire resistant. According to Dutch and British tests this ensures a fire resistance of one hour. Greater fire resistance, when required, results of course from a thicker layer of protective mortar.

If, except for the anchorages, no ties existed at all between the tendons and the web, the ultimate strength of the beam would be smaller than if the tendons were fully bonded to the concrete. But the ultimate strength of the beam is easily, cheaply and fully restored by the provision of a few connections between the tendons and the web; for example in the form of pieces of thin reinforcing bars that stick through the web between the layers of wires, and are embedded in mortar together with the wires. Any connection, be it ever so slight, between the tendons and the concrete web is sufficient to prevent lateral buckling of the latter due to the prestressing tendons being external.

External tendons afford several important advantages. They eliminate the ducts and all that goes with them: the cost and trouble of buying and installing them, the impediment to the concreting of the often thin web, the reduction of the shear strength of the web, sometimes the necessity of increasing the web thickness for concreting purposes even though the shear strength may not require such an increase, and last but not least, the grouting operation. One can seldom be perfectly sure that grouting will leave no voids or pockets of water in the duct, especially when steel tubing is used, which prevents the dispersal of water through the concrete, and there is no easy way to ascertain their absence. The mortar covering external cables, on the contrary, can easily be inspected, and removed and replaced locally if it should prove faulty. Furthermore, cracking of beams after grouting in winter, even several weeks after grouting, has occurred in many instances. This risk is eliminated by external cables.

Tying precast blocks together to form prestressed beams can also be done more conveniently with external tendons.

The author voices the opinion that external tendons solve many post-tensioning problems and that they can be used to advantage in a great many cases. Their use however, is sometimes precluded by aesthetic considerations.

The Sainctelette Viaduct in Brussels (designed by Willems and Fougnies) provides one example of statically determinate members that are prestressed by external tendons. It consists of 50 almost identical, simply supported spans, most of them 81 ft. 6 in. long (fig. 1). Although the viaduct has a 30 ft. wide

Fig. 1 BRUSSELS VIADUCT
LONGITUDINAL SECTION OVER ONE SPAN

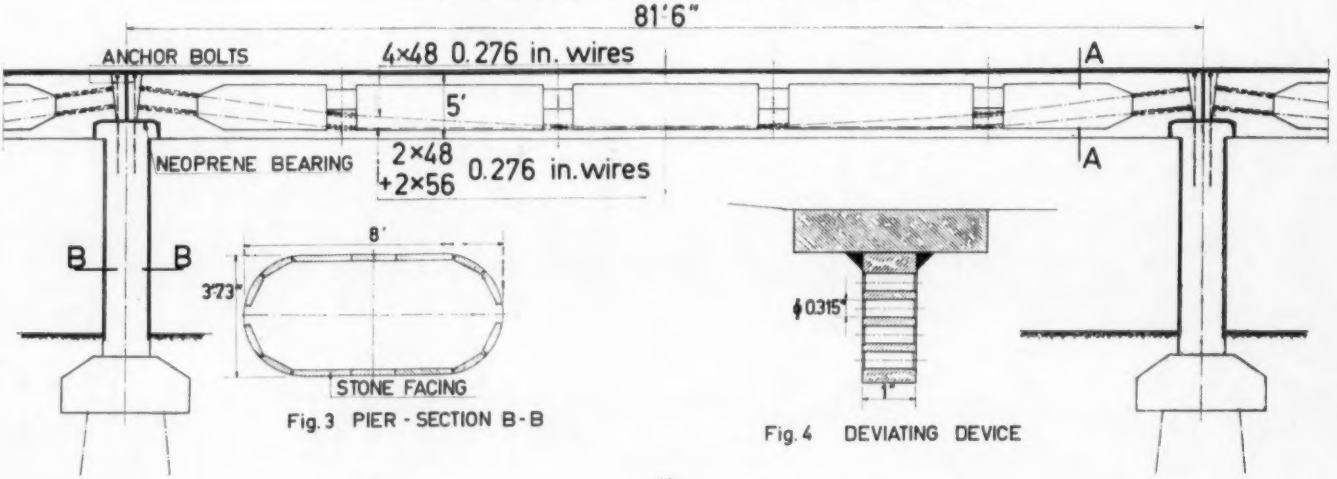


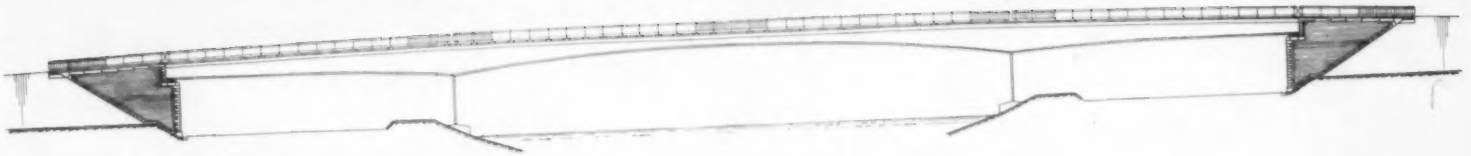
Fig. 3 PIER - SECTION B-B

Fig. 4 DEVIATING DEVICE

Fig. 2 SECTION A-A

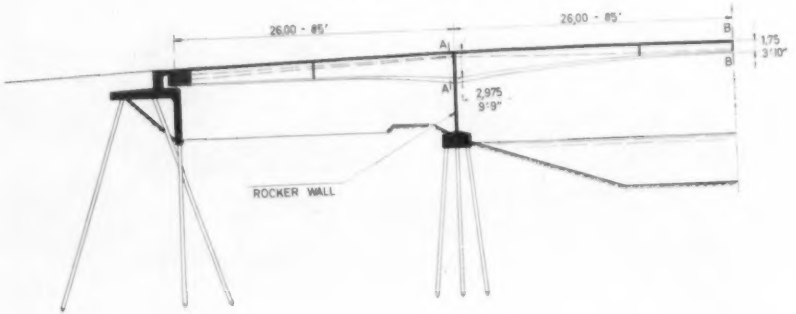
Fig. 5 BRIDGE W18

ELEVATION

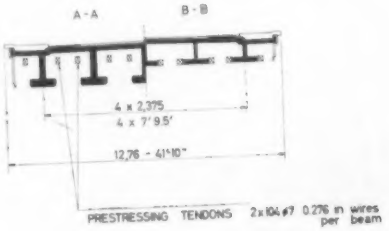


1/2 LONGITUDINAL SECTION

Fig. 6



CROSS - SECTION



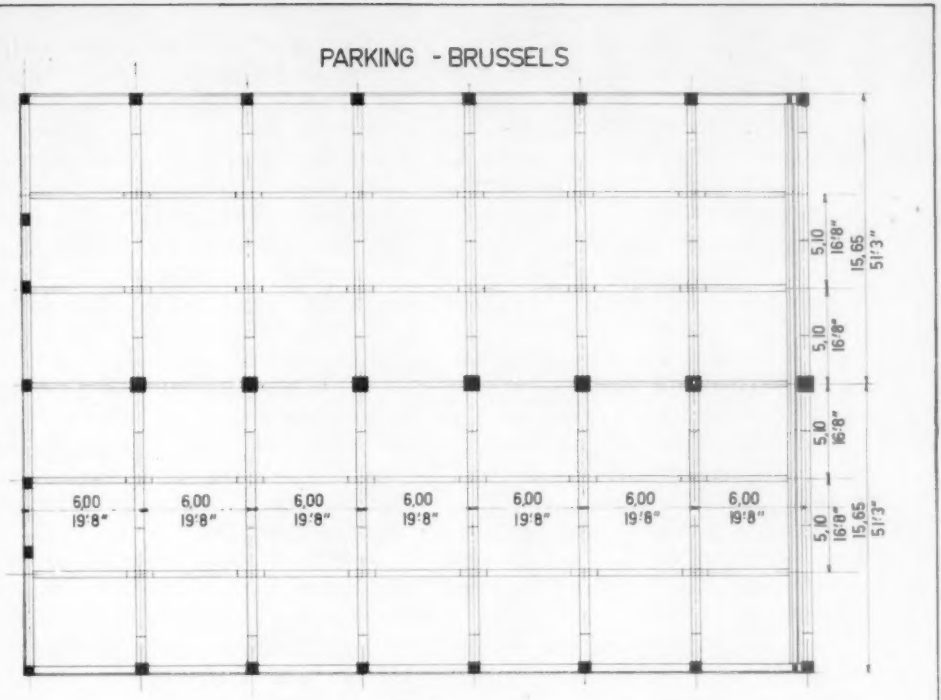


Fig. 7

roadway and a total width of nearly 39 ft. (fig. 2), it rests on piers only 8 ft. wide (fig. 3), made this narrow for aesthetic reasons and also to leave more room for the tramways on the street level.

The superstructure comprises a 5 ft. deep box with sloping flanks and two cantilevering slabs. The eight polygonal prestressing tendons, some made up of 48 and others of 56 0.276 in. wires, are installed inside the box. The simple deviating devices (fig. 4) bear against transverse concrete diaphragms.

The prestressed wires are encased in mortar inside the box girders. Ventilation of the interior of such girders is an absolute necessity. It is provided here by holes in the bottom slabs.

Four bridges built across the "*Ringcanal*" in the vicinity of *Ghent* are of the type shown in fig. 5 and 6. The bridges also span two roads that follow the canal banks. The piers are comprised of footings which carry concrete rocker walls that are only 12 in. thick.

The main carrying members of the roadway are continuous prestressed concrete beams. They were cast on falsework, successively the lower flange, the web and the roadway slab, with shear keys along the seams between them. There are two external tendons per beam, one on either side of the web. Each is composed of 104 wires of 0.276 in. diameter. The small rectangles shown in the cross-section (fig. 6) represent the mortar in which the tendons are embedded.

Parking building in Brussels (Lipski).

Figure 7 shows the plan of a typical floor of the main part of this building. The three rows of columns are spaced about 50 ft. apart, and the columns in each row are at 19 ft. 8 in. intervals. The main beams are prestressed and extend transversely across the building. Together with the three columns they constitute a rigid frame (fig. 8). They support secondary reinforced concrete beams at 16 ft. 8 in. intervals. Each main beam is made up of factory-precast blocks, as shown in fig. 8. There are eight sections to each span. The cross-section of the end block is rectangular, that of five other blocks is I-shaped. Two special short blocks are provided at the intersections of the main and secondary beams.

After the concrete for the center columns was cast at least up to the level of the upper fiber of one of the main beams, the sixteen blocks composing this beam were aligned on a simple scaffolding. The end columns did not extend above the preceding floor during this phase of the erection. The joints between the blocks and between the center column and the two adjacent blocks were then grouted with cement mortar. Two prestressing tendons, each of twenty 0.276 in. wires, were then threaded, one on either side of the web, through the holes provided in the end blocks, the special blocks and the center column. After the grout in the joints had been allowed to harden for three or four days, the cables were tensioned, and anchored at the ends in the usual way. The scaffolding was then removed, except for supports under the end blocks, and the tendons were embedded in mortar (fig. 8 sec. B).

The concrete for the end columns, the secondary beams and the floor slab was then cast in forms. A slice of the 4 in. thick slab extends across the upper flange of the main beam. The beam and the slab are tied together by sloping stirrups. Continuity of the secondary beams over the main beam is achieved by means of reinforcement which protruded from the special blocks provided at their intersections. Continuity between the end columns and the main beam is achieved by means of reinforcing bars which are embedded partially in the column and partially in the floor slab. The total depth of the completed floor is slightly less than 2 ft., except in the vicinity of the columns, where the main beams are haunched.

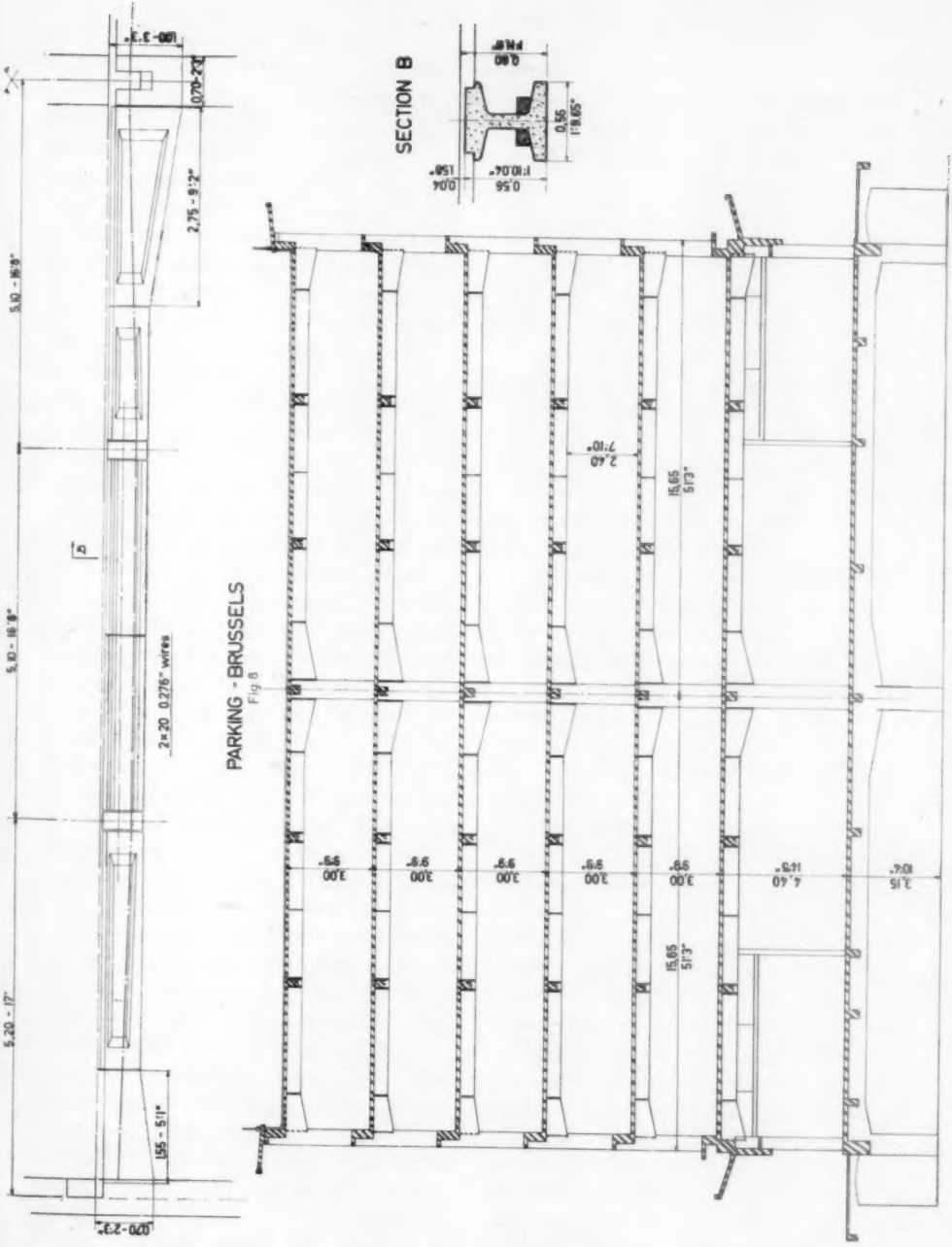
By this method, prestressing was combined with precasting for a structure that is practically monolithic and highly statically indeterminate in its final form. Continuity is a virtual necessity, of course, in the case of a multi-storey building. The erection procedure described was used mainly to avoid excessive bending in the end columns.

Methods of Achieving Continuity of Prefabricated Beams.

Several methods have been devised to combine the advantages of the prefabrication not merely of concrete blocks, but of complete prestressed beams, with those of continuity.

One method, which originated in France, consists of placing and temporarily supporting the prefabricated prestressed beams between the supports (columns or main beams). Then the joints are grouted, and so-called "cap cables" are threaded through ducts provided in the intermediate supports and in the end parts of the prefabricated beams, and these cables are tensioned.

Another method, which originated in Belgium, is designed to elimi-


SECTION B
PARKING - BRUSSELS
Fig 8
2×20 0.276" wires

nate the risk of errors involved in tensioning very short, curved cables and the anchoring of them by means of devices based on friction. It replaces the cap cables by straight 0.8 in. or 1 in. diameter screwed rods. These bars have a breaking strength of about 170,000 lb. per sq. in., and they are stressed up to about 92,000 lb. per sq. in. and anchored by means of nuts. In a factory at Charleroi, beams about 50 ft. long, were prestressed on the ground and then hoisted into position in rows of seven units each. These beams were thus transformed into continuous beams with a total length of about 350 ft. The intermediate supports are either columns or main beams supported by these columns. With either of the methods described, the prefabricated members carry their own weight as simply supported beams, and all other loads as continuous beams. The shear between the ends of the prefabricated beams and the supports is carried by the friction which is developed concurrently with the tension in the cap cables or in the threaded bars. Both methods can, of course, be used for transforming columns and prefabricated beams into rigid frames as well as for making prefabricated beams continuous over intermediate supports.

A less versatile method has been used in the construction of several bridges in the Netherlands. The end parts of the prefabricated beams are narrower than their upper flange is elsewhere, and they extend beyond the pier. By the stressing of transverse tendons the beams are tied together and friction forces them to deflect together, thus making them continuous over the pier.

The three methods described can obviously be used to achieve continuity on the site between prefabricated members prestressed by pretensioning as well as between members prestressed by post-tensioning.

Adjustments of Supports.

Adjusting the external connections of a statically indeterminate structure may lead in some cases to a more favorable distribution of the bending moments. For example, by pushing one leg of a two-hinged portal frame inwards, one can cause the dead load bending moment at the center of the span to vanish, if so desired, or to take any desired value. Such adjustments can be performed on steel or reinforced concrete structures as well as on prestressed concrete structures, of course, but the idea derives from the same general concept as the principle of prestressing. In fact, adjustment of the external connections of a structure does for the external forces acting on it what prestressing does for the internal forces. Adjustment of supports probably was first used in conjunction with prestressing in France. The flat jack, invented by Freyssinet, can be a powerful tool in carrying out such an operation.

One important fact about such adjustments, when applied to concrete structures, must not be overlooked. The stress distribution in an indeterminate structure does not depend on the modulus of elasticity of the concrete, if the supports do not move from the position they had while the structure was unstressed. Hence, creep of the concrete does not affect the stress distribution, provided that the modulus of elasticity may be considered as a constant throughout the structure at any given moment. This does not hold true when relative displacements of the supports occur, whether they be ac-

cidental or created intentionally. The bending moments produced by such displacements are proportional to the modulus of elasticity, and hence decrease as time goes by; they may fall off to less than half of their initial value; they decrease much more than the stresses created by prestressing tendons. The designer should either allow in the analysis for that large decrease, or, if he relies on the full value or almost the full value of the additional bending moments in order to ensure a certain moment distribution, he should make provision for later readjustment of the supports, thereby allowing the builder to bring the external reactions up to their design value.

Heusden Bridge (Lipski—fig. 9, 10 and 11)

The reactions of the superstructure of this bridge on its end supports are always directed upwards, whatever the position of the live load. Therefore, each end of the superstructure is connected, by means of prestressed concrete anchors, to two reinforced concrete slabs, which are loaded with sand. Each anchor has a large reinforced concrete button at its upper end (fig. 11) and two flexurally weak sections, thus functioning as a movable support.

After the tensioning of the cables in the superstructure and removal of the falsework, the pull in each anchor amounted to 35,000 lb. An additional traction of about 30,000 lb. was then created in each anchor by pushing down both ends of the roadway and by blocking them in their new position. This operation was designed to decrease the permanent bending moment in the middle portion of the center span.

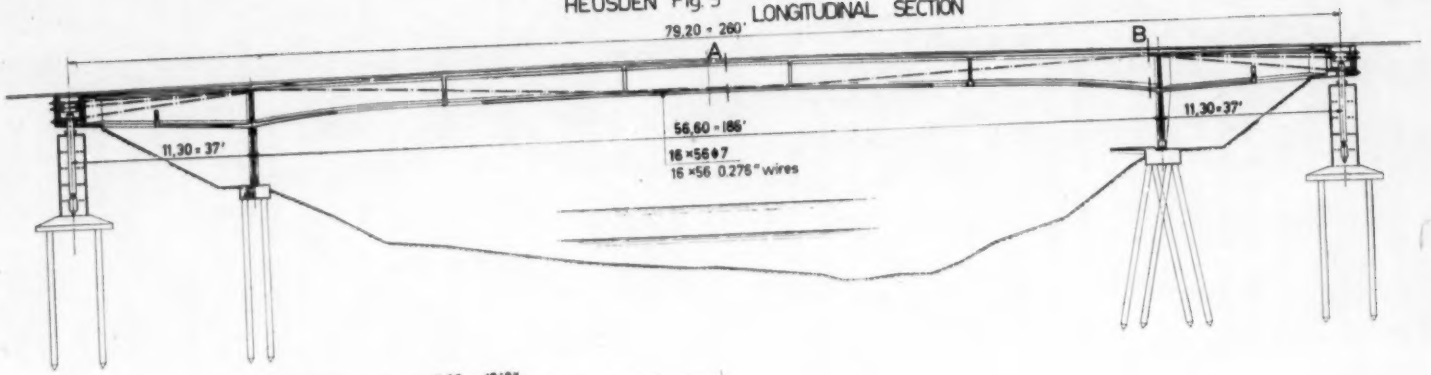
It remains possible to check at any time the magnitude of the pull in the anchors, and to bring it back to its design value when necessary by increasing the thickness of the blocking between the anchor buttons and the bottom of the recesses in the ends of the superstructure.

Construction of Prestressed Concrete Bridges Without Falsework.

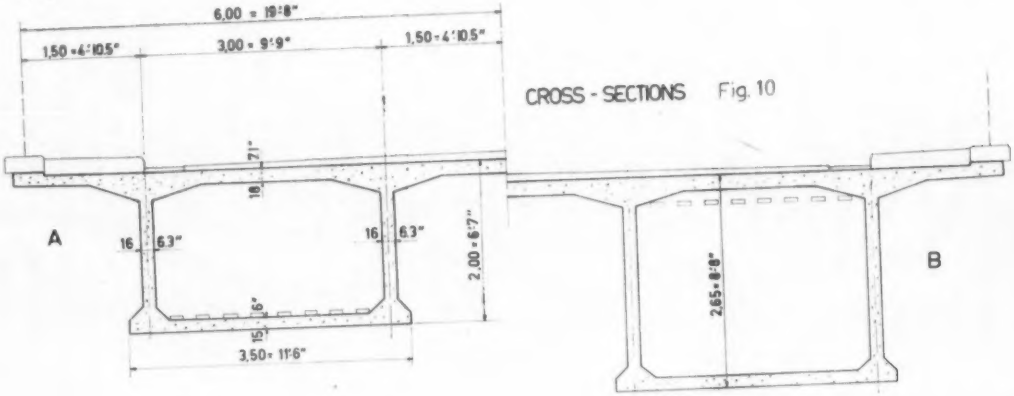
Several large prestressed concrete bridges were built across rivers in Germany by gradually cantilevering out the superstructure from the piers or abutments. Falsework was thus done away with. This remarkable achievement enables prestressed concrete to encroach further on what could once be considered as the exclusive domain of structural steel. It was made possible by the development of prestressing tendons that can conveniently be made up of relatively short sections.

Construction of the superstructure starts from the abutments and, symmetrically, from the piers, if any. If the abutments are not strong enough to withstand overturning by the lengthening cantilevers, trestles are erected at some distance from the abutments to support the cantilevers temporarily. The superstructure is constructed in sections 10 or 12 ft. long. The prestressing tendons consist of screwed, high strength rods, that are as long as the sections of the deck structure and are spliced together by means of sleeve-nuts. A carriage that cantilevers out from and is supported by the last finished section carries the shuttering for the next section. One more rod is added to each prestressing bar, save a few bars that may end at the finished section. The concrete for the new section of the superstructure is then poured. When it has acquired sufficient strength, it is tied to the preceding section by the tensioning of a few bars, which may either end there or be length-

HEUSDEN Fig. 9 LONGITUDINAL SECTION



CROSS - SECTIONS Fig. 10



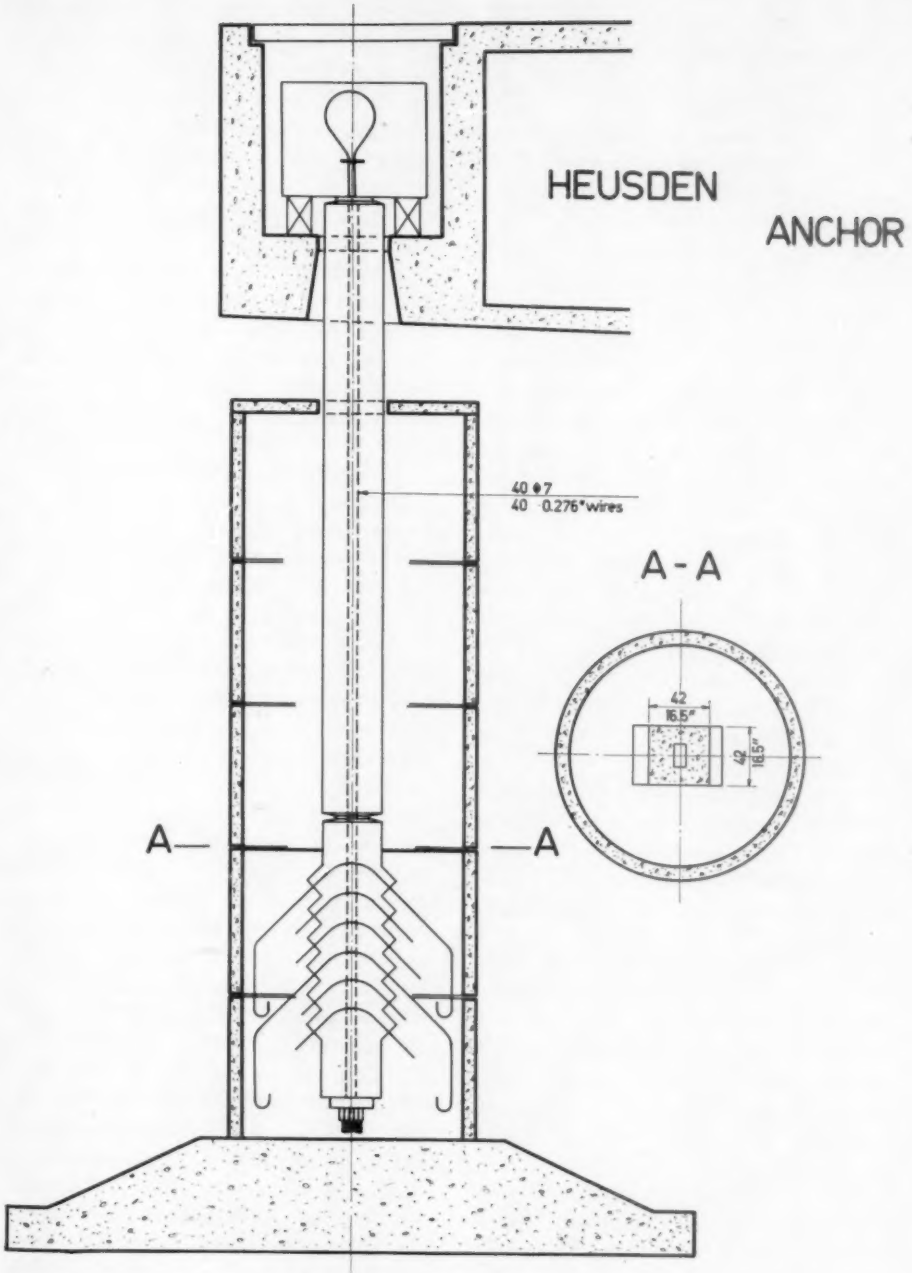


Fig. 11

ened again later into the next section. The carriage is then moved forward once more over the length of one section and the cycle is started all over again. Some tendons are not stressed before the two cantilevers meet at or close to the center of the span. These unstressed bars are subsequently made continuous across the seam between the two cantilevers, which are then made continuous themselves by the tensioning of these bars.

The load conditions existing during the successive construction stages influence the distribution of the prestressing steel to a large extent. For that reason, the total amount of steel to be provided usually exceeds the amount required in the finished structure. It should be noted that the corresponding extra expenditure for this steel sometimes does not cancel out completely the saving on the scaffolding.

Casting transverse diaphragms would be a nuisance when construction is carried out by the cantilever method. Bridges built by this method are therefore designed without such diaphragms. This is in keeping with a fairly general trend toward reduction of the number and possibly toward elimination of transverse diaphragms in the design of prestressed concrete deck bridges, even of bridges that are built on falsework.

The *Dischinger bridge in Berlin* is one of the bridges that was built by the cantilever method. The main carrying system is a large frame supported by caisson foundations.

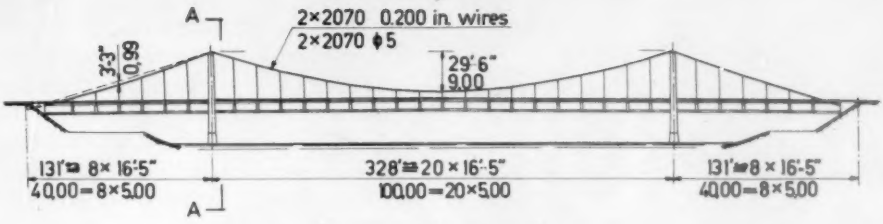
Two box girders of variable depth, a slab joining them, and two cantilevering slabs make up the cross-section. There are no transverse diaphragms in between the abutments. Prestressing steel is provided in the form of high strength rods of about 1 in. diameter.

A 17 ft. long section of the superstructure adjoining each abutment was cast first on scaffolding. Construction then proceeded by the cantilever method in sections of 12 ft. length. Equilibrium of the first five sections was insured by vertical tie bars stretched between the end of the superstructure and the caisson. Trestles were then erected about 65 ft. from the abutments and served as temporary supports while construction of both halves of the superstructure continued simultaneously until they met at the center of the span. On an average, five days were needed to complete a 12 ft. long bridge section.

Prestressed Concrete Suspension Bridges.

The concept of the prestressed concrete suspension bridge was explained at length in a paper presented at the 1957 World Conference on Prestressed Concrete in San Francisco. What was said there will not be repeated here. It may be recalled simply that a bridge of the type discussed here behaves structurally as a bridge which is prestressed by cables with a large eccentricity and also as a self-anchored suspension bridge with concrete instead of steel stiffening girders. This type of bridge enables engineers to design economical highway suspension bridges in the range of spans of, say, 200 to 350 or 400 ft., that are too short for steel suspension bridges. On the other hand, the prestressed concrete suspension bridge also opens to prestressed concrete the otherwise barely accessible field of spans above, say, 400 ft. As the span length increases, the steel suspension bridge becomes its most direct competitor, to which it is greatly superior so far as rigidity is concerned. It is definitely feasible

Fig.12



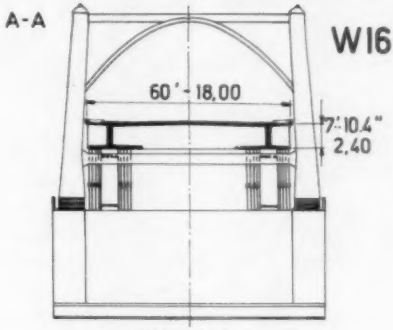


Fig.13

technically and economically to build prestressed concrete suspension bridges of spans as long as 1,600 ft.

The elevation of the superstructure of the Mariakerke bridge is reproduced in figure 12, and that of one pier and of the reinforced concrete tower it supports, together with the cross-section of the suspended deck structure, is reproduced in figure 13. It was convenient to cast the concrete for the superstructure of this bridge on scaffolding, for the canal crossed by the bridge was not dug at the time of its construction. But a bridge of this type can also be built without falsework, high above deep water when necessary.

The two main cables are parts of one endless strand, which loops around the two ends of the bridge deck. This skein was built up by the placing of the wires, untensioned, around the ends of the bridge, in the cable bands and in the saddles, while the frames supporting the saddles were 28.3 in. below their design position. Prestressing of the entire superstructure was effected by pushing up alternately one tower and then the other with respect to the piers, until they reached their design elevation. The eight 300-ton jacks acting on each column exerted a total force of 4,960,000 lb. at the end of the operation. A prestressing force of 12,340,000 lb. was thus produced in the deck structure. The economy of this type of bridge is due to the relative smallness of this force for a bridge of 328 ft. span and 60 ft. width, and to the slenderness of the deck.

It may be mentioned, incidentally, that a concrete suspension bridge can be prestressed in other ways than the one chosen at Mariakerke, for instance by pulling on both cables at one end of the bridge, thus copying the usual post-tensioning

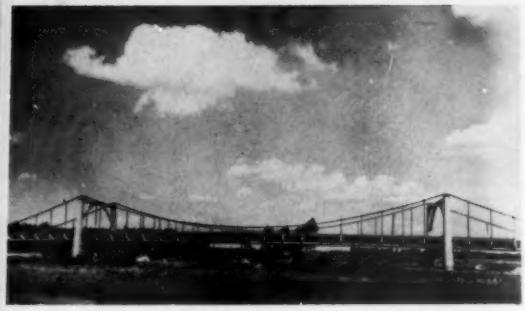


Fig. 14

procedure, or by casting the floor structure above its design elevation and by lowering it after the cables are installed, the towers being concreted in their design position in either case.

The Mariakerke bridge was subjected to a number of tests. One consisted in placing between the towers 29 vehicles with an aggregate weight of 1,600,000 lb. This loading produced a maximum deflection of 3.0 in., equal to 1/1,300 of the span, and a maximum permanent deflection of 0.22 in. As shown by these figures and by the results of the other tests, the Mariakerke bridge possesses ample rigidity.

Figures 14 and 15 show the finished structure.

Prestressed Concrete Shells.

A considerable number of prestressed concrete shells were constructed in Europe during the last decade, mainly in Great Britain, Germany and the Netherlands. But the author will describe only one shell structure, which was made up almost entirely of precast elements.

Philips pavilion

Among the many unusual structures at the 1958 Brussels World's Fair, one of the most daring, though not one of the largest, was the Philips pavilion.

Its maximum height was about 67 ft. (fig. 16) and its maximum length, measured across its plan, was about 90 ft. (fig. 17). The surface of the structure consisted of a number of hyperbolic paraboloids which intersected along straight lines. The geometrical surfaces were materialized by prestressed concrete shells composed of 2 in. thick precast slabs and their intersections were comprised of prestressed cast in situs, circular concrete ribs 16 in. in diameter.

It is quite impossible to analyse such a structure and therefore the working stresses in and the buckling strength of the shells were evaluated by means of tests on models. The level of prestress in the structure was chosen so as to eliminate all tensile stresses.

The pavilion rested on a pile-supported reinforced concrete beam, which followed its contour. In the initial construction stages the lower edge of the shells was free to slide with respect to the foundation, layers of bituminous paper being interposed to that end between the curved foundation beam and the rake-like edge beam of the shells (fig. 18). The teeth of the rake were tied, vertically only at first, to the foundation, to prevent possibly excessive rotation of the edge of the thin walls due to imperfectly centered prestressing forces or to bending moments which disturbed the membrane state of stress in the shells.

Fig. 15

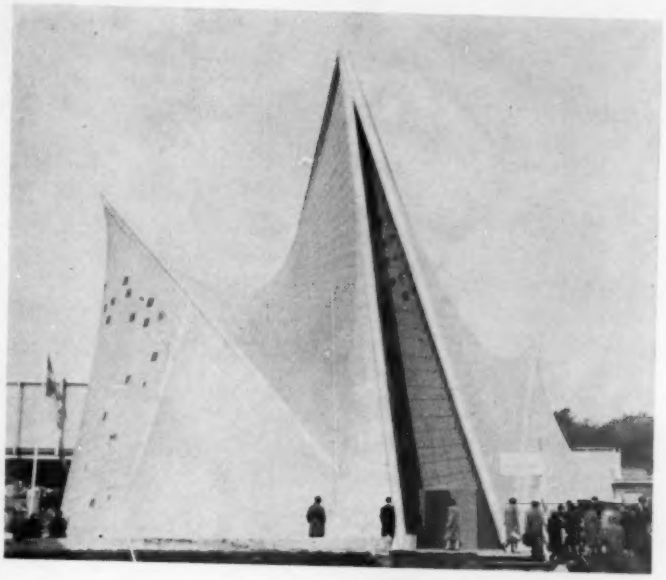


Fig. 16

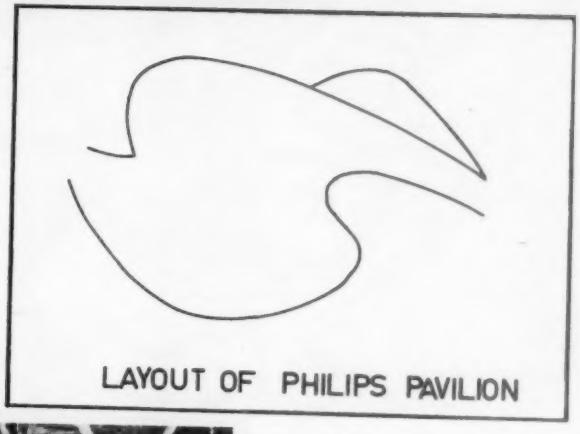


Fig. 17

Fig. 18

Fig. 19

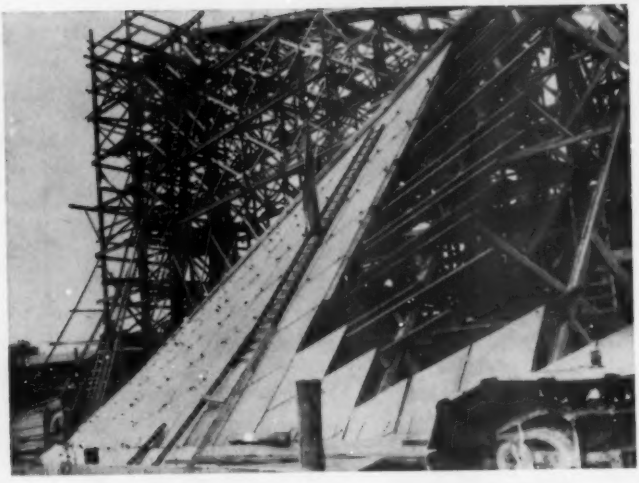


Fig. 20

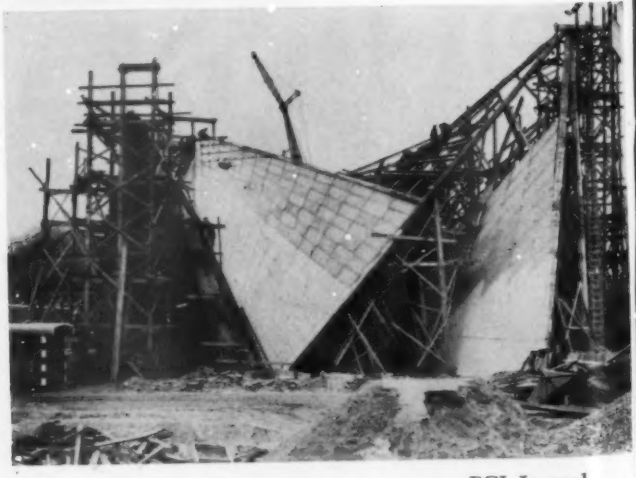


Fig. 21

The precast, 2 in. thick slabs were cast on sand forms (fig. 19). These forms were moulded simply by the help of strings which represented one family of generatrices of the hyperbolic paraboloid and slid along straight boards representing generatrices of the other set. The joints between the slabs also materialized generatrices and were therefore straight. No two slabs were identical.

After the edge beam and the cylindrical ribs had been cast in situs and after some of the 0.354 in. diameter lubricated wires cast into the ribs had been tensioned, the slabs were erected on a scaffolding (fig. 20 and 21) and the joints between them caulked with mortar. Prestressing was then started in the shells and gradually and simultaneously carried on in all the ribs and shells. The shells were prestressed by means of 0.276 in. wires strung against and tied to both the inside and the outside faces of the prefabricated slabs, their ends being attached to the ribs. Tensioning was carried out by means of a screw jack placed perpendicularly to the wires (fig. 22). As far as possible the wires were stretched along generatrices of the hyperbolic paraboloids. Thus there was no loss of stress in the wires through friction, and the wires would stay close to the wall even if they were not tied to it. The ties were, of course, still indispensable to safeguard the shells against buckling.

The prestressing of the building caused slight displacements of the lower edge of the shells over the foundation beam. When all wires were tensioned, concrete was cast between the teeth of the rake-like edge beam and encased the mild steel bars projecting from the foundation (fig. 18). Further movements of the lower edge of the shells were thereby prevented. Finally the scaffolding was removed. The prestressing wires on the outside and inside faces of the building were painted, this protection against corrosion being deemed adequate because of the temporary character of the pavilion. The author does not suggest that this structure was economical, but it did illustrate the remarkable strength of shells with double curvature and the versatility of the combination of precasting and prestressing.

Fig. 22

Proposed Footbridge.

The author also wants to mention for the record a footbridge proposed for, but not actually constructed at the 1958 World's Fair in Brussels. It would have been an expensive freak structure, such as are only built on the grounds of world's fairs. Its deck, although spanning 130 ft., would not have been more than 6 in. thick.

Miscellaneous Structures.

Many more or less unusual applications of the principle of prestress-

ing could be added to the foregoing examples, such as the prestressed foundations of the television tower in Stuttgart and of the Hermes Tower in Hannover, the prestressed tie members between the springings of the huge shell roof of the Exhibition Palace in Paris, prestressed concrete tie members and suspenders for bowstring bridges, prestressed concrete roof trusses built in Great Britain, prestressed tunnel linings, underpinning of mediaeval towers at Tournai, Belgium, and prestressed suspended roofs, for example the roof of the Benjamin Franklin Hall in Berlin. The author does not enlarge upon these and many other topics here, partly because some of the applications of prestressing involved are rather obvious or relate only to a minor part of the structure, partly because some of the solutions adopted are not likely to be used frequently, partly because some of the structures mentioned are well known in the United States or are discussed in other papers presented at the Convention, and finally because the present survey can not possibly be complete.

Readers' Comment

DISCUSSION

"TENTATIVE RECOMMENDED PRACTICES FOR GROUTING POST-TENSIONED PRESTRESSED CONCRETE"

By Ben C. Gerwick, Jr.
President, Ben C. Gerwick Inc.

Gentlemen:

The Committee is to be highly commended for the excellence of their work which should fill a much needed void (no pun intended) in current American specifications.

The following items are suggestions, or questions, from my limited experience and have probably been previously considered by the Committee.

Paragraph 103.4-"Aluminum powder . . . . . *should* be added, etc." Perhaps *may* would be a better word than *should*. My understanding is that aluminum powder is not universally used in Europe nor in the United States.

Paragraph 104.1(B)-Last sentence: "Metallic sheaths should be of a *ferrous* material." We have used and specified galvanized sheaths (zinc), and have heard that lead coated sheaths have been used to reduce friction.

Paragraph 106(A)-I have never observed this sequence used in mixing, but presumably it represents an improvement developed by grouting specialists.

Paragraph 106(D)-The water cement ratio of 5½ gallons per sack of cement looks too wet. I understand the best European practice calls for a W/C of .35 or .45 maximum, which is somewhat lower than the 5½ gallons per sack = .49.

Paragraph 107(A)-I believe ducts should be flushed out with water and then blown clear with compressed air. If water is not used, then there is a possible problem of excessive absorption of water from the grout. If the flushing is not fol-

lowed by blowing clear with compressed air, then the chances of water being trapped are increased.

Paragraph 108.2-Is the grout compressive strength of 2500 psi at seven days high enough? I think the grout strength should be the same as the specified concrete strength of the member itself.

Paragraph 108.3-Typographical error in title—change "Trail" to "Trial".

Discussion by Grouting Committee on Ben C. Gerwick, Jr. comments on "Tentative Recommended Practices for Grouting Post-Tensioned Prestressed Concrete".

Paragraph 103.4-The word "should" has been used as it does not indicate that the requirement is mandatory. Mr. Gerwick's comment is probably correct that aluminum powder is not universally used in the United States. However, the committee feels that it should be universally used.

Paragraph 104.1(B)-We agreed with Mr. Gerwick that the last sentence in this paragraph is incorrectly stated. A better sentence would be "Sheaths should be of a material which shall not deteriorate and which shall not react with any other materials in the unit". This is a change we would put in a later rewriting of the entire recommended practices.

Paragraph 106(A)-It is the opinion of the committee that the sequence of mixing as listed will prove to be the more satisfactory in the greater number of cases.

Paragraph 106(D)-It is the opinion of the committee, particularly with long small diameter ducts in hot weather where the ducts are cored rather than formed by tubing, that a water cement ratio of 5½ gallons per sack will be required. Other parts of the specifications place maximum limits on shrinkage and minimum limits on strength. We feel that with these limitations a proper grout will be made.

Paragraph 107(A)-We have not specified that the ducts shall be flushed with water. This was omitted as the experience of various members of the committee indicated that where ducts are formed of a water tight material it is not necessary that the duct walls be wetted prior to grouting. We also have not specified that if the ducts are flushed that they shall be blown out with compressed air. There is considerable evidence, including demonstration tests made with clear plastic ducts, that better grouting may be done if the grout is pushing water ahead during the injection process. This point is controversial and we were unable to determine enough conclusive evidence to indicate that the water either should or should not be blown out prior to grouting.

Paragraph 108.2-The committee's opinion was that where long small diameter ducts are in use that a grouting with a compressive strength of 2500 PSI may be the most that can be obtained. It is our opinion that a grout which does not shrink is of greater importance than one of high strength. The main purposes of the grout are, of course, to develop bond and to prohibit corrosion of the steel. Tests indicate that a 2500 PSI strength is satisfactory for bond. The use of aluminum powder is necessary to counteract shrinkage. This reduces compressive strength somewhat in the same proportion as does the use of other air entraining agents. We feel that the use of aluminum powder to reduce the shrinkage outweighs resulting reduction in strength.

Paragraph 108.3-The typographical error should be changed.

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Past President Peter J. Verna presented a bronze plaque to Col. Martin P. Korn at the Fifth Annual PCI Convention. This plaque symbolized the establishment of the annual Martin P. Korn Award.

The Martin P. Korn Award of the Prestressed Concrete Institute was established at its Fifth Annual Convention at Miami Beach in honor of Martin P. Korn, first Executive Secretary of the Institute. This Award will be presented annually to the author of the best paper published in the PCI JOURNAL.

COMPETITION RULES

I Competition for the Martin P. Korn Award is restricted to members of the Institute.

II The Award consists of $100.00 in cash, with an appropriate engraved certificate. In case of more than one author, the cash will be divided, but each will receive a certificate.

III All original papers presented to the Institute by members in any grade, and published in the JOURNAL during the year for which the Award is given, are open to the Award provided that such papers have not been previously contributed in whole or in part to any other associations, nor have appeared in print prior to their publication by the Institute. Papers written jointly by members and non-members are not eligible. Papers to be considered for this year's Award are those that appear in the December 1960, March, June and September 1961 issues.

IV The Award, with certificate, is awarded to the author, or authors of a paper which shall be judged worthy of special commendation for its merit as a contribution to the advancement of prestressed concrete. The ceremony conferring the Award will be held at the 1961 PCI Convention in Denver, Colorado, Oct. 15-19.

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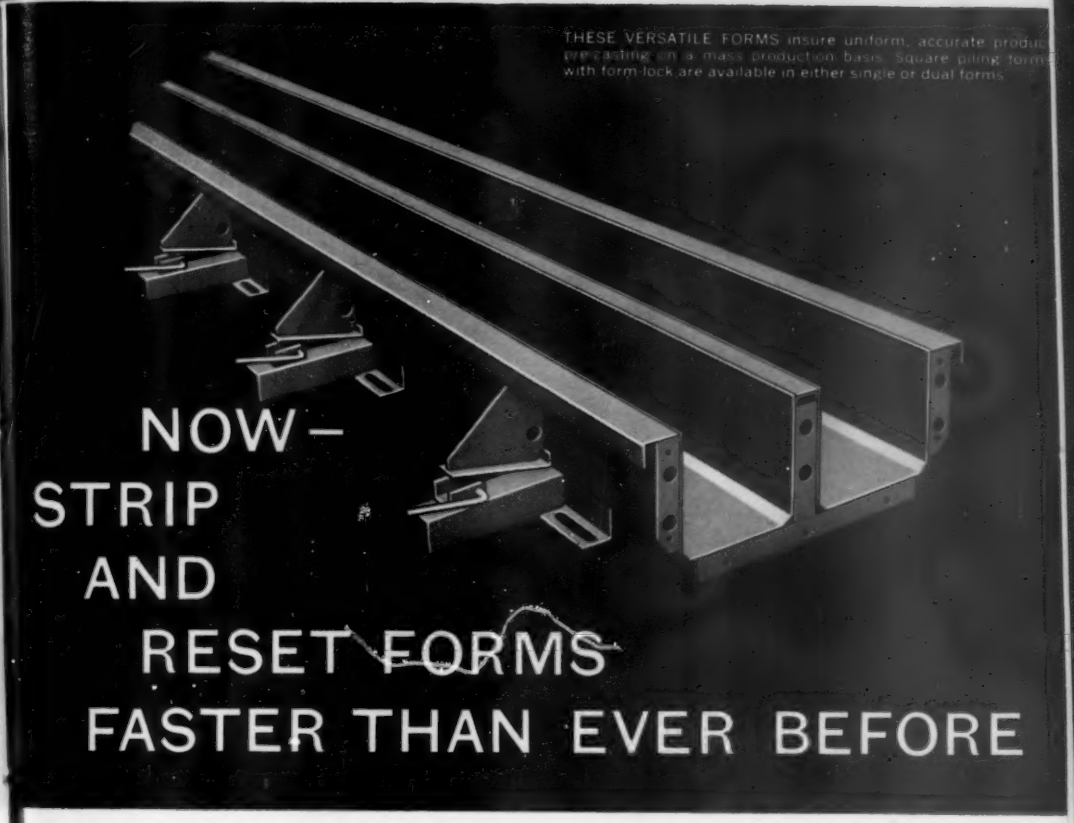
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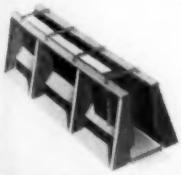

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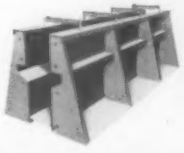

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Aerial view of bridge. Note railroad siding built expressly for handling bridge beams. As work progressed, river was diverted and siding laid over dry bed. Bridge designed by the Nebraska State Department of Roads. John Hossack, Acting State Engineer; G. C. Strobel, Assistant Acting State Engineer; Art Dederman, Bridge Engineer; General Contractor, Massman Construction Company, Kansas City, Missouri. Prestressed concrete members by Nebraska Prestressed Concrete Company, Lincoln, Nebraska.

LOW BIDDER USES PRESTRESSED CONCRETE FOR MAJOR BRIDGE PROJECT

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